US Army Corps of Engineers
Waterways Experiment Station

August 1994

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Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Proceedings of REMR Workshop on Levee Rehabilitation

compiled by E. B. Perry

WES

Approved For Public Release; Distribution Is Unlimited

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Prepared for Headquarters, U.S. Army Corps of Engineers

The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

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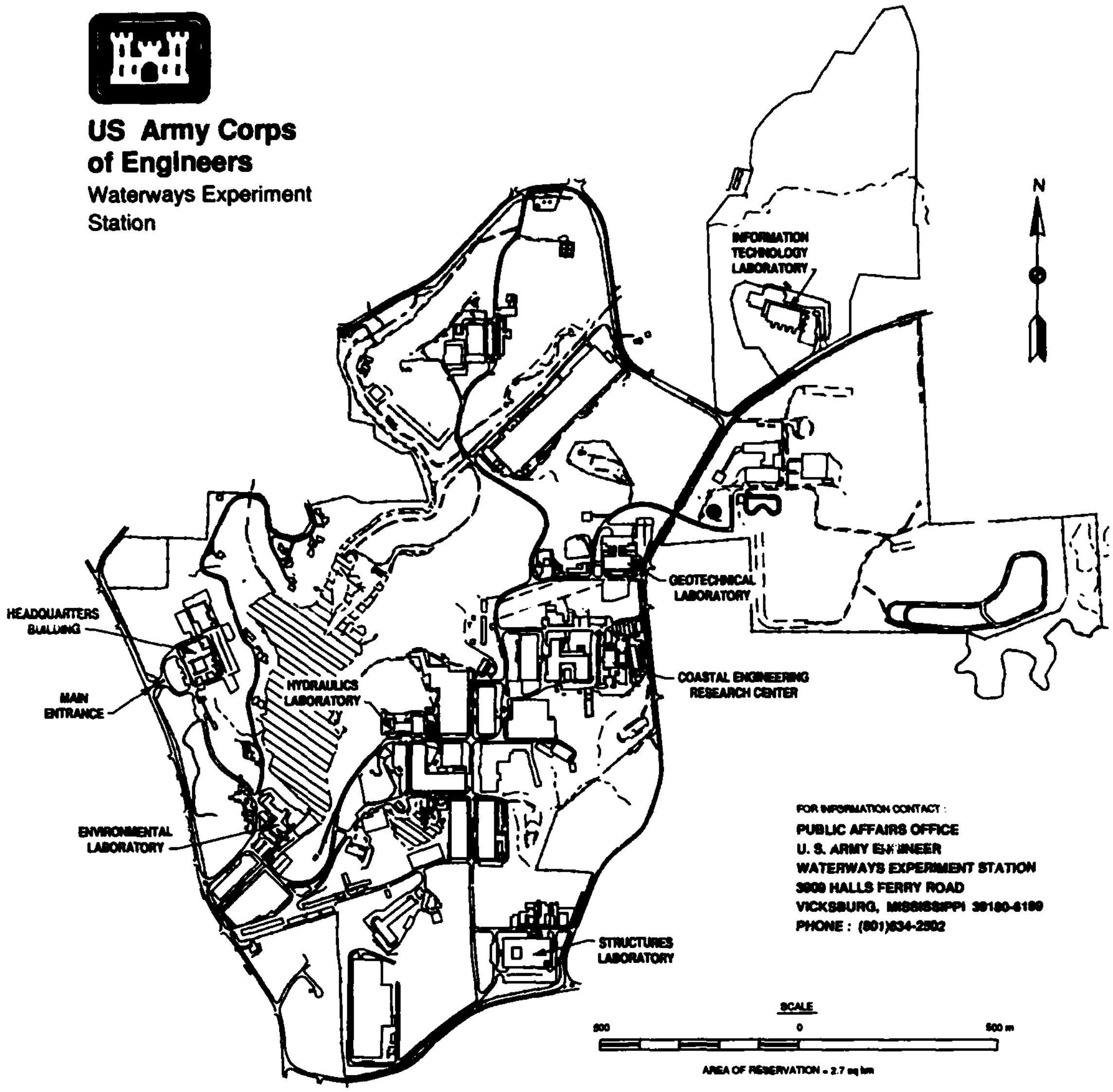
Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

US Army Corps of Engineers

Waterways Experiment Station



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Contents

Preface

The Proceedings of the Repair, Evaluation, Maintenance and Rehabilitation (REMR) Research Program Workshop, "Levee Rehabilitation," were prepared for the Headquarters, U.S. Army Corps of Engineers (HQUSACE), by the U.S. Army Engineer Waterways Experiment Station (WES).

The workshop was conducted under REMR Work Unit 32646, "Levee Rehabilitation." Mr. William N. Rushing (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE; Mr. James E. Crews (CECW-O) and Dr. Tony C. Liu (CECW-EG) served as the REMR Overview Committee. Mr. Arthur H. Walz, HQUSACE, was Technical Monitor for this work. Mr. William F. McCleese, WES, was the REMR Program Manager, and Mr. Gene P. Hale, Geotechnical Laboratory (GL), WES, was the Problem Area Leader.

This workshop was organized by Dr. Edward B. Perry under the general supervision of Dr. Don Banks, Chief, Soil and Rock Mechanics Division, GL; and Dr. William F. Marcuson III, Director, GL.

During publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
Fahrenheit degrees	5/9	Celsius degrees or kelvins[1]
feet	0.3048	meters
gallons (U.S. liquid)	3.785412	cubic decimeters
inches	2.54	centimeters
pounds (force) per inch	175.1268	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (mass)	0.4535924	kilograms
square inches	6.4516	square centimeters

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the *following formula*: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

Introduction

The Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Workshop on "Levee Rehabilitation" was held at the U.S. Army Engineer Waterways Experiment Station (WES) on 17 March 1992. The workshop was sponsored by REMR Work Unit 32646 entitled "Levee Rehabilitation."

The purpose of the workshop was to stimulate exchange of ideas and information regarding innovative methods for levee rehabilitation, directions for analytical and laboratory research, and possible field demonstrations of innovative methods.

The workshop was attended by 17 people. A list of attendees is given on the following page. Presentations were made on seismic damage to levees, lime stabilization of levee slides, use of geogrids for levee slope repair, use of rock-fill trenches to stabilize levees, use of geotextiles for levee construction on soft soils and soil nailing for slope repair. A copy of available written lectures is included in these Proceedings.

ATTENDEES
REMR Workshop on Levee Rehabilitation
Vicksburg, Mississippi 17 March 1992

Name	Organization	Phone No.
Dennis Abernathy	Memphis District	(901) 544-3381
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George Postol	St. Louis District	(314) 331-8399
Laura Rowland	Memphis District	(901) 544-4016
Gerry Satterlee	New Orleans District	(504) 862-1000
George Sills	Vicksburg District	(601) 631-5631

AGENDA
REMR Workshop on Levee Rehabilitation
Classroom No. 2, Building 1006
Waterways Experiment Station, Vicksburg, Mississippi

Time	Presentation	Speaker
8:00 am	Welcome and Intro to Workshop	Edward B. Perry Waterways Experiment Station
8:15	Overview of REMR Work Unit 32646 on Levee Rehabilitation	Edward B. Perry Waterways Experiment Station
8:30	Seismic Assessment of Pajaro and San Lorenzo River Levees After Loma Prieta Earthquake	Leon Holden San Francisco District
9:15	Lime Stabilization Slide Repair at Bardwell Lake Embankment	Douglas Massoth Ft. Worth District
10:00	Break	
10:15	Double Lime Application for Levee Stabilization	Mark S. Alvey St. Louis District
11:15	Use of Geogrids for Levee Slope Stability Problems	Dennis W. Abernathy Memphis District
12:00	Lunch	
1:00	Lime Stabilization and Rock-Fill Trenches	George L. Sills Vicksburg District
1:45	Lime-Fly Ash Injection of Levees	Jerry A. Holloway Kansas City District
2:30	Break	
2:45	Levees Constructed on Soft Soils using High Strength Geotextiles	Philip J. Napolitano* New Orleans District
3:15	Use of Soil Nailing for Slope Repair	Gerard S. Satterlee New Orleans District
3:45	Questions and Discussion	All
4:30	Adjournment	

* Presented by Gerard S. Satterlee.

Overview of REMR Work Unit 32646 on Levee Rehabilitation

Edward B. Perry
Waterways Experiment Station

Introduction

The Corps of Engineers is responsible for 8,500 miles[1] of levees. Levees are subject to overtopping, current and wave attack on the riverside slope, surface erosion of slopes and crest resulting from rainfall, through-seepage causing softening and sloughing of the slope in the vicinity of the landside toe and associated piping problems, underseepage resulting in uplift pressures on the landside impervious top stratum with associated sand boils and piping problems, and slope instability in the form of deep-seated or shallow surface slides. In many cases, conventional methods of levee rehabilitation are both costly and time consuming. For example, when levees are located in urban areas, the expense involved in obtaining necessary rights of way for conventional rehabilitation measures, such as landside seepage berms for control of underseepage is prohibitive leaving innovative methods as the only feasible solution. Table 1 shows conventional and innovative methods of levee rehabilitation for the various types of damage outlined above.

Overview of Work Unit

The objective of this work unit is to develop guidelines for applying innovative chemical and physical techniques to levee rehabilitation. The work unit will consist of identification of innovative chemical and physical techniques, a laboratory test program to develop required data such as effects of fly ash and lime injection on the changes in strength of plastic clays with time, analytical studies such as three-dimensional slope stability analysis of mechanically

[1] A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page v.

stabilized shallow surface slides, field demonstrations of appropriate innovative systems, and development of guidelines for applying innovative chemical and physical techniques to levee rehabilitation. The results of this study will consist of input to the engineering manuals, a REMR technical note on levee rehabilitation, and a final report. Utilization of the innovative chemical and physical techniques developed in this study will result in better, more economical levee rehabilitation methods.

Purpose of Workshop

The purpose of this workshop is to foster interchange regarding innovative methods for levee rehabilitation, indicate directions for analytical and laboratory research, and suggest field demonstrations of innovative methods.

Table 1
Conventional and Innovative Rehabilitative Methods for Various Levee and Floodwall Problems

	Rehabilitative Methods	
Problem	**Conventional**	**Innovative**
Overtopping	• Rebuild • Vegetation • Concrete Blocks • Precast Post and Panel Wall	• Lightweight material to raise levee to prevent overtopping • Inflatable dams/water structures/geotubes • Geotextile-reinforced grass geocells
Current and Wave Attack	• Vegetation • Revetment (riprap, concrete rubble, articulated concrete mattress, gabions)	• Geotextile-reinforced grass revetment (used auto tires, soil cement blocks, etc.)
Surface Erosion	• *Vegetation* • Chemical stabilization	• *Geosynthetic systems*
Through Seepage	• *Pervious toe drain*	• Geocells (with vegetation)
Underseepage[1]	• Conventional cutoffs • Riverside blankets • Landside seepage berms • *Pervious toe trench* • *Pressure relief wells*	• Slurry trench cutoffs • Jet grouted cutoffs
Slope Instability	• Drainage • Remove and replace soil (slope flattening and benching) • Conventional Restraint • Structures • Chemical treatment	• Mechanically stabilized soil nailing with geotextiles • Fly ash and lime injection • Gravel trenches • Slide Suppressor walls • Vegetation[2] • Micro (root) piles • Continuous polymer thread injection

[1] Covered under REMR-I Work Unit 32274.
[2] Covered under REMR-II Work Unit 32644.

Seismic Assessment of Pajaro and San Lorenzo River Levees After Loma Prieta Earthquake

Leon Holden
San Francisco District

Introduction

The 7.1 magnitude Loma Prieta Earthquake which occurred on 17 October 1989 caused major damage to the San Lorenzo River levee system and the Pajaro River levee system. Damage consisted of levee subsidence and small to large cracks developing along the crest, on the slope, and along the land-side and riverside toe of the levee. Cracks also developed within berm areas and away from the protected side toe of the levee. Most of the damage were the result of liquefaction of the foundation materials. There were also damage areas that showed signs of lateral movement.

The most severe damage to the levees occurred near the mouth of the rivers. The levee reaches near the mouths contained liquefiable foundation materials and a high water table.

In reaches where lateral movement was thought to be the primary cause of damage, there were no liquefiable materials detected. However, a weak cohesive layer of saturated material was detected. There were no obvious signs of crest subsidence, but very long, deep and wide cracks did develop. Visible cracks up to 12 in. wide, several feet deep and hundreds of feet long were observed and recorded.

The San Francisco District, U.S. Army Corps of Engineers awarded a contract to Granite Construction Company for emergency repair work to the levees. Because repairs were made under emergency conditions, no subsurface soils investigation and geotechnical analyses were performed to determine the condition of the levees. A scope of work was approved and funding provided through the Emergency Operations Branch to perform a post earthquake assessment of the levee system.

Background

The San Lorenzo River and the Pajaro River levees were designed by the U.S. Corps of Engineers, San Francisco District. The soils reports presented in the General Design Memorandum did not address the consistency of the foundation materials nor the seismicity and potential liquefaction of the area. During the time of the levees design, liquefaction was not widely known and was not considered in the analyses and design of levees. Earthquake or seismic loading was considered in the slope stability analyses for the San Lorenzo River levee. A seismic coefficient of 0.1 was used in the pseudo-static seismic method for the most critical arc resulting from the static long term slope stability analyses.

The levees were constructed with excavated materials from the channel improvement works. No improvement was done to the foundation. The Pajaro River levees were constructed in 1949. The San Lorenzo River levees were constructed in 1959. The Loma Prieta Earthquake liquefied the foundation materials and caused major damage to the levee embankment within selected reaches.

Location

The San Lorenzo River levee system is located along the San Lorenzo River in the City of Santa Cruz, California. The Pajaro River levee system is located along Pajaro River and Salsipuedes Creek near Watsonville, California. Figures 1 through 3 show the location of the two levee systems.

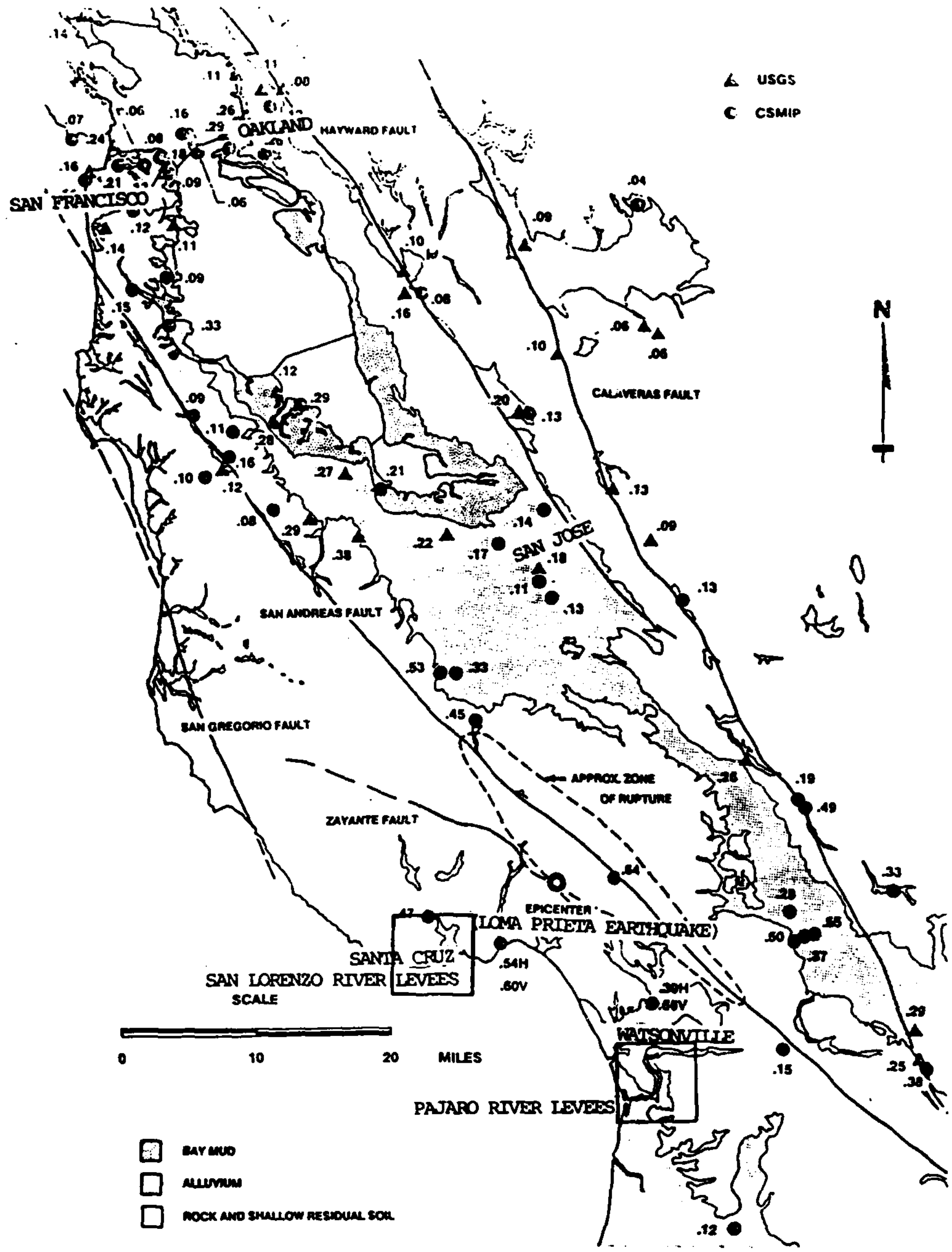
Earthquake Damage

San Lorenzo River Levee

The earthquake caused severe cracking and deformation of the levee embankment. Approximately 3 to 4 ft of subsidence occurred in the levee crest. Large cracks up to 6 in. wide and several feet deep developed along the slope and crest of the levee within the lower reaches. Less damage occurred in the upper reaches. Pumping stations and drainage structures in the lower reaches were also damaged. Photos of the damage are shown in Exhibit A. A summary of the damage by reach is presented in Table 1.

Pajaro River Levee

Damage occurred throughout the levee system on both the Santa Cruz County and Monterey County sides of Pajaro River. The most severe damage occurred in the lower reaches near the mouth of the river. The Loma Prieta



Overview of Recorded Peak Horizontal Ground Surface Accelerations During the Loma Prieta Earthquake of October 17, 1989

Figure 1. Location map

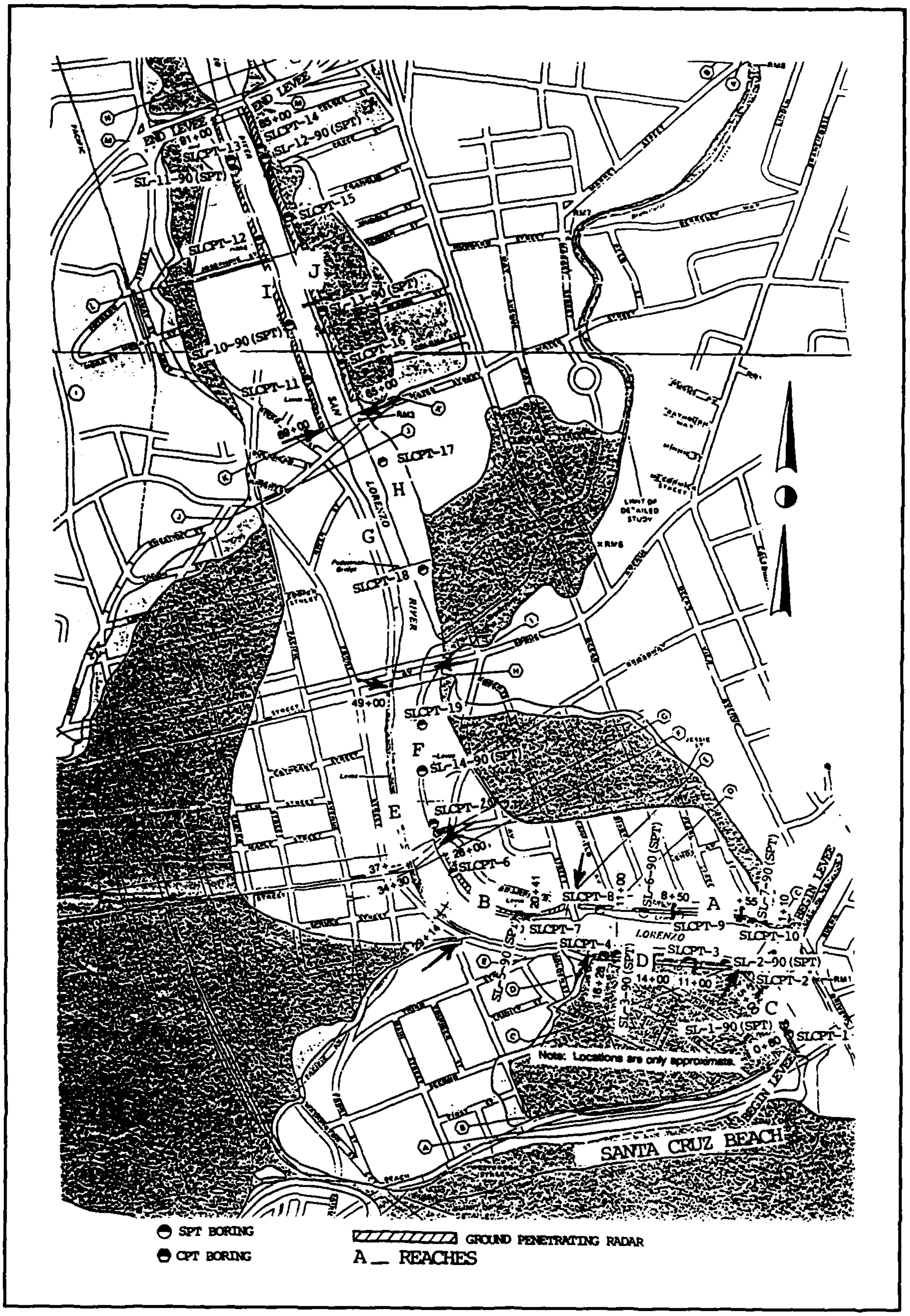


Figure 2. San Lorenzo River site

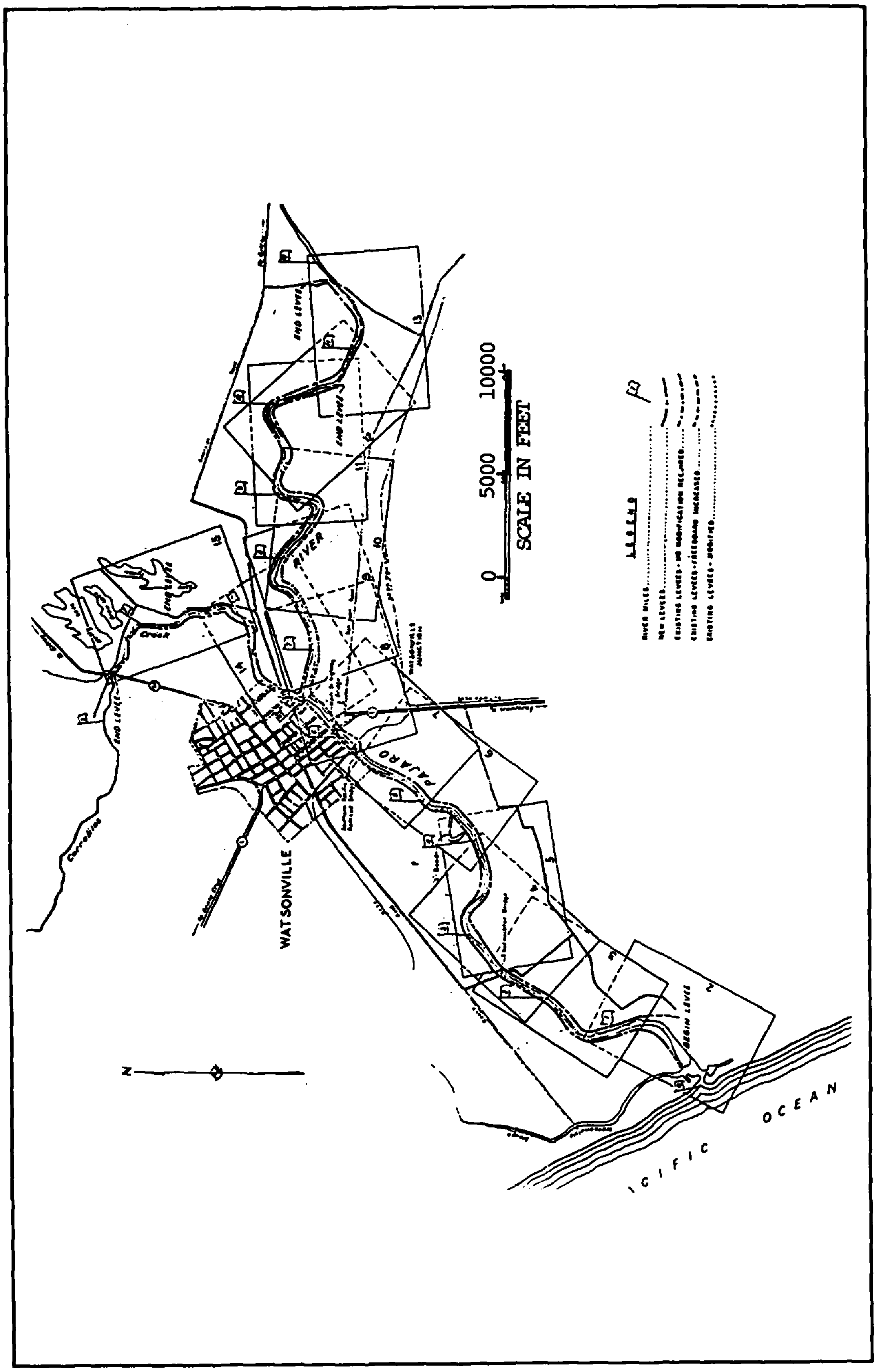


Figure 3. Pajaro River site

Table 1
San Lorenzo River, Summary of Post Earthquake Assessment

Reach	Degree of Earthquake Damage	Type of Repair	Condition of Foundation	Method of Strengthening Foundation (Cost)	Approximate Cost (Strengthening)	Approximate Cost of 1989 Repair[1]
A (1500 ft)	Slope and crest cracking and deformation. Severe.	Excavated and recompacted embankment to 6 ft maximum depth.	Weak liquefiable materials throughout reach. Approximately 12 ft thick layer.	(a) Vibrocompaction ($900,000) (b) Compaction grouting ($820,000)	$820,000	$500,000 (35% of $1.4 million)
B (1200 ft)	Slope and crest cracking and deformation.	Excavated and recompacted embankment to 5 ft maximum depth.	Approximately 9 ft thick layer of liquefiable material within 700 ft reach.	Compaction grouting ($250,000)	$250,000	$140,000 (10% of $1.4 million)
C-D (1600 ft)	Slope and crest cracking and deformation.	Excavated and recompacted embankment to 6 ft maximum depth.	10 ft thick layer of liquefiable material within reach.	(a) Vibrocompaction ($900,000) (b) Compaction grouting ($800,000)	$800,000	$560,000 (40% of $1.4 million)
E (2000 ft)	Cracks developed along edge of pavement at crest on protected side.	Excavated and recompacted embankment to 5 ft maximum depth.	Pressurized sewer located within embankment. No drilling was done. 20 ft to 25 ft thick liquefiable layer assumed.	Compaction grouting.	$320,000	$150,000
F (1000 ft)	No significant damage.		Foundation appears adequate against liquefaction.	None		
G	None	None	No levee/no drilling.	None		
H	None	None	No levee.	None		
I (2300 ft)	None. No visible surface damage	None	Foundation appears adequate against liquefaction.	None		
J (3280 ft)	Minor cracks appeared in crest at upstream end of reach.	Excavated and recompacting (mainly patch work).	Foundation appears adequate against liquefaction (sewer located within embankment).	None		

[1] The 1989 emergency repair work cost approximately $1.4 million. Needed repairs at the pumping stations were estimated to cost $221,000. *According to CESPN* report titled "Post Earthquake Assessment, San Lorenzo Levee System, for Damaged Structural Features," 29 November 1989.

earthquake cause major damage and revealed the locations of liquefiable soil and low shear strength soil sites throughout the levee system. Exhibit A shows photos of typical earthquake induced damages to the levees.

Large cracks developed in the crest, slopes, and along the toe of the levee. Most of these cracks were in the longitudinal direction. There were some locations where the cracks occurred in the transverse direction. The location and degree of damage caused by the earthquake are presented partially in Table 2.

Emergency Repair Work

The San Francisco District contracted to make emergency repairs to the damaged levees along the San Lorenzo River and the Pajaro River levee systems. Repairs consisted of excavating and recompacting up to 6 ft of embankment material in the severely damaged areas. Imported compacted fill was added to bring the repaired section back to grade. In areas where there were no apparent subsidence, the cracks were repaired by shallow excavating and recompacting the soil. Exhibit A shows typical repairs made to the damaged levees following the earthquake. Structural repairs were also made at pumping stations and gravity outlet works.

Post Earthquake and Repair Subsurface Investigation

A subsurface investigation was performed to determine the condition of the levee embankment and foundation. Standard Penetration Test, SPT, borings; Cone Penetration Test, CPT, probings; and Ground Penetrating Radar, GPR, geophysical subsurface data were obtained during the investigation.

Standard Penetration Test, SPT

Pajaro River Levee System. Forty-five (45) SPT borings were drilled from 10 to 20 September 1990. The 6 in. diameter borings were drilled through the top of the levee into the foundation. The borings were drilled to a depth of 36 ft using a truck mounted rotary wash drill rig. Relatively undisturbed samples were recovered in cohesive materials with the Dames and Moore Type U sampler. Blow counts, N, and soil samples were obtained using 1.375-in. I.D. SPT sampler at intervals ranging from 2.5 ft to 10 ft. The sampler was driven in accordance with ASTM method D-1586.84 using a 140-lb hammer and 30 in. free fall.

San Lorenzo River Levee System. Eleven (11) SPT borings were drilled from 16 to 24 April 1990. The borings were drilled through the top of the levee into the foundation to a depth of approximately 4 ft using rotary wash

Table 2
Sample Pajaro River Levee Post Earthquake Assessment, Santa Cruz County

Levee Stationing (Approx.)	Degree of Damage	Emergency Repair Method	Geotechnical Assessment
	Santa Cruz County		
47+67-63+92 (1,625 ft)	U.S. end of earthquake damaged reach (47+67 to 63+92), *major cracks 2 ft wide and 8 ft visible depth* crest settlement.	Excavated and recompacted.	Boring PASPT-4 station 63+00±, hard clay to silt to 10 ft, soft and firm clay from 10 to 20 ft, *very stiff clay from 20 to* 25 ft, medium dense silty sand from 26 to 33 ft, very stiff clayey silt to 40 ft
	No reported earthquake damage.		Cone PACPT station 74+00±, very soft clay layer at depth of 19 to 21 ft, very weak clayey to silty sand layer at depth of 14.5 to 22 ft, weak layer at depth from 26 to 28 ft (silt), very soft to soft clayey material from 30 to 50 depth. Approximately 6 ft thick sandy silt to silty sand layer from 21.5 to 28.5 ft.
	No reported earthquake damage.		Boring PASPT-5 station 107+00±, clayey silt to 24 ft, very soft at 15 ft, loose silty sand to silty clayey sand to 36.5 ft.
122+12-122+15 (3 ft)	Small en-echelon transverse cracks.	Trench.	
123+62-126+00 (238 ft)	A distress crack has developed in Riverside Levee slope. The crack was inspected on 10 April 1991. The crack is approximately 150 ft *long, up to 9 in.* wide, and has a visible depth of ±5 ft. 238-ft adjacent upstream section damaged during earthquake. Oblique transverse crack (5 ft deep).	Excavated and recompacted.	Cone PACPT-4 station 123+65±, silt to silty sand in upper 10 ft, very soft clay to silty clay from 14 to 19.5 ft, loose silty sand from 20 to 26 ft, soft clay to clayey silt from 26 to 33 ft and from 35 to 38 ft, very soft to soft clay from 39 to 48 ft. Boring PASPT-6 station 124+90±, dense silty sand to 10 ft, stiff clay from 10 to 19 ft, firm clayey silt from 19 to 25 ft, dense clayey silty sand from 25 to 30 ft, stiff clay to 36.5 ft.
	Monterey County		
34+47-37+18 (271 ft)	Longitudinal cracks up to 3 ft wide.	Excavated and recompacted.	PASPT-38, station 41+10, very dense to dense silty sand to 10 ft, stiff sandy silt from 10 ft to 18 ft, dense sand from 18 to 25 ft, *soft clay from 26 to 30 ft*, dense silty sand from 30 to 34 ft, *very soft clay layer 34 to 35.5 ft*, stiff sandy silt to 36.5 ft.
38+54-46+70 (816 ft)	Longitudinal cracks at crest and sides, up to 5.5 ft visible depth and 1.0 ft wide, transverse cracks with 2 in. vertical offset.		PACPT-31, station 47+20, dense sand to 5 ft, *soft to very soft silty clay from 5 to 17 ft, very soft clayey silt layer from 29 to 31 ft*, soft clay layer from 38 to 43 ft, non-liquefiable sandy silts and silty sands from 17 to 29 ft and from 31 to 38 ft, medium dense silty sand from 43 to 47 ft.

(Continued)

Table 2 (Concluded)

Levee Stationing (Approx.)	Degree of Damage	Emergency Repair Method	Geotechnical Assessment
Monterey County (Continued)			
50+93-52+85 (192 ft)	Longitudinal cracks on crest and sides, 5.5 ft deep, 6 in. wide.	Excavated and recompacted.	PACPT-30, station 53+00, sand to medium sandy clayey silt to 7 ft, firm to soft clay from 7 ft to 16 ft, clayey silty sand to medium dense sand from 17 to 29 ft, silty clay from 29 to 43 ft, silt to clayey silty sand from 43 to 50 ft.
56+97-61+43 (446 ft)	Transverse and longitudinal cracks at crest.		PACPT-29, station 57+50, sandy silt to loose silty sand to 19 ft, *liquefiable* silty sand from 13 to 19 ft, medium dense sand from 19 to 27 ft, very soft to soft silty clay from 27 to 30 ft, soft to firm clay from 31 to 41 ft, firm sandy silt to dense sand from 41 to 50 ft.
64+47-67+83 (336 ft)	Longitudinal cracks up to 4 ft visible depth in crest, cracks in sideslope up to 6.5 ft visible depth.	Excavated and recompacted.	

and a Failing 1500 drill rig. A 6-in. diameter triton bit was used to advance the boring to a depth of 10 ft. A 6-in. diameter casing was set, and a 4.5-in. diameter bit and drilling mud were used to complete the boring. Below counts, N, and soil samples were obtained at intervals ranging from 2.5 to 10 ft using the 1.375-in. I.D. SPT sampler. The sampler was driven with a 140-lb hammer in accordance with ASTM method D-1586-84.

Cone Penetration Test, CPT

Fifty-three (53) CPT probings were performed along the Pajaro River Levees and the San Lorenzo River Levees from 13 to 21 September 1990. Eighteen (18) probings were completed along the San Lorenzo River Levees and thirty-five (35) probings were completed along the Pajaro River Levees.

CPT probings were performed in accordance with ASTM method D 3441-86 using a Hogentogler electronic cone. The equipment used consisted of a cone penetrometer, a series of hollow rods, a set of hydraulic rams mounted in a truck to provide thrust of up to 20 tons and a data acquisition system. The cone had a conical tip with a 60-deg apex angle and cylindrical friction sleeve. The cones were equipped with internal piezometers for measuring soil pore water pressure and inclinometers for measuring cone verticality. The cones had cross section areas of 10 sq cm, and a sleeve surface of 150 sq cm. Transducers within the cone allowed for simultaneous measurement of cone and sleeve resistance; pore pressure and inclination during penetration. *Samples of the CPT plots are shown in* Figures 4 through 8.

Geophysical Subsurface Profiling

Ground penetrating radar, GPR, subsurface profiling was performed in selected reaches in order to determine the relative homogeneity within the levee embankment and to detect any major anomalies. A SIR 3 GPR system manufactured by Geophysical Survey Systems, Inc. was used to collect the profiles. The GPR transmits a radio frequency electromagnetic pulse into the ground. The pulse travels through the ground at approximately 2 to 3 nanoseconds per foot. Three pulse frequencies were used during the data collection. The frequencies used were 900 MHz, 500 MHz, and 300 MHz. The higher frequency provided sharper resolution of shallow depths. The lower frequency provided greater depth of record but less sharpness of resolution.

GPR records were obtained at damaged-repaired locations and at undamaged-unrepaired locations. GPR subsurface profile records for the Pajaro River Levees indicate nonhomogeneity of the undamaged-unrepaired levee embankment. Samples of the GPR records are shown on Figures 9 through 13. A comparison of records obtained at the damaged-repaired site and at the undamaged-unrepaired sites shows a significant difference in the relative homogeneity of the levee embankment materials. Discontinuities can be seen in the profile of the soil layers at the unrepaired sites. This is typical of the more than 19,000 lin ft of records collected along the levees.

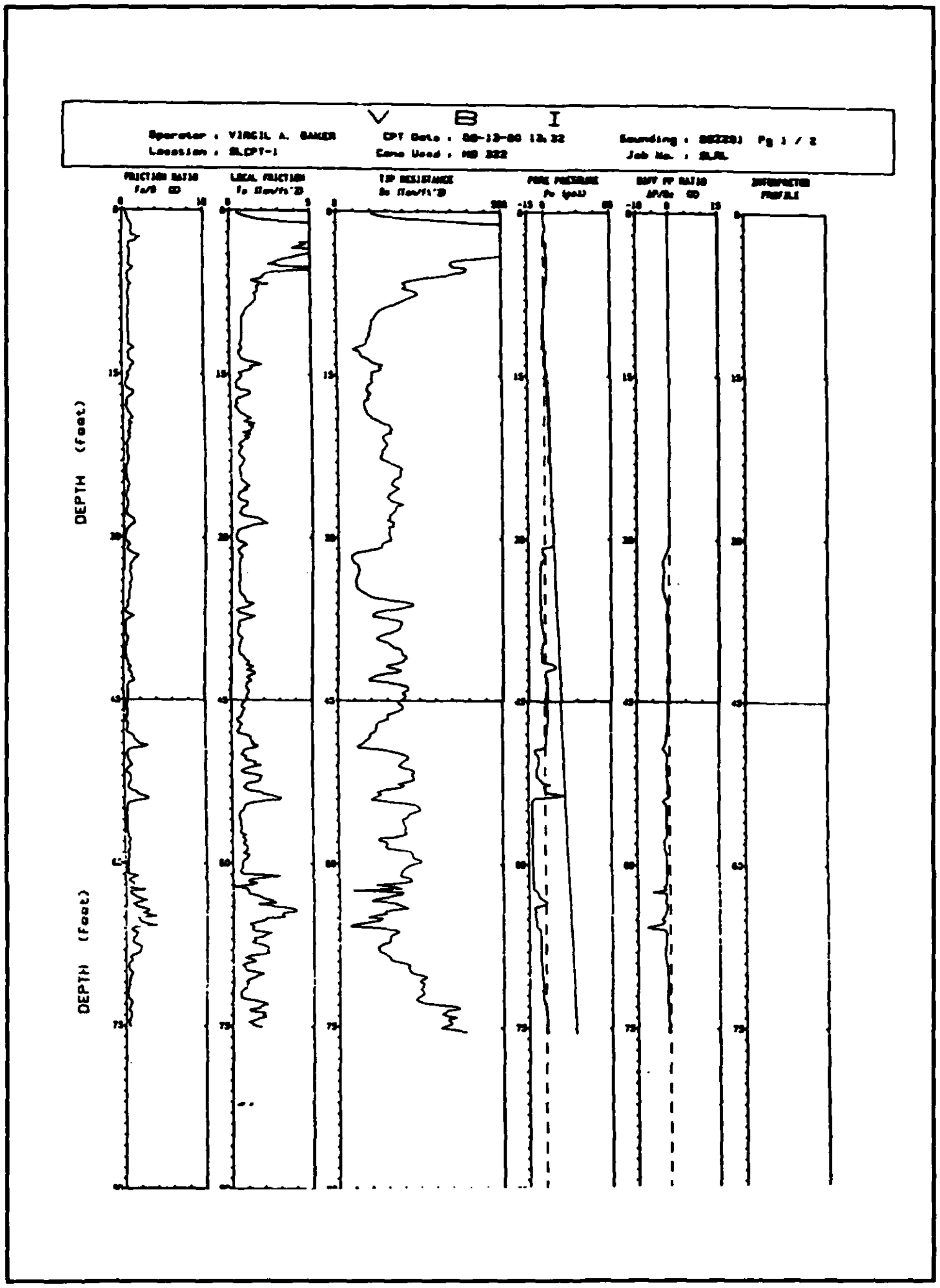


Figure 4. Sample CPT plot San Lorenzo River levee (see Figure 2 for location)

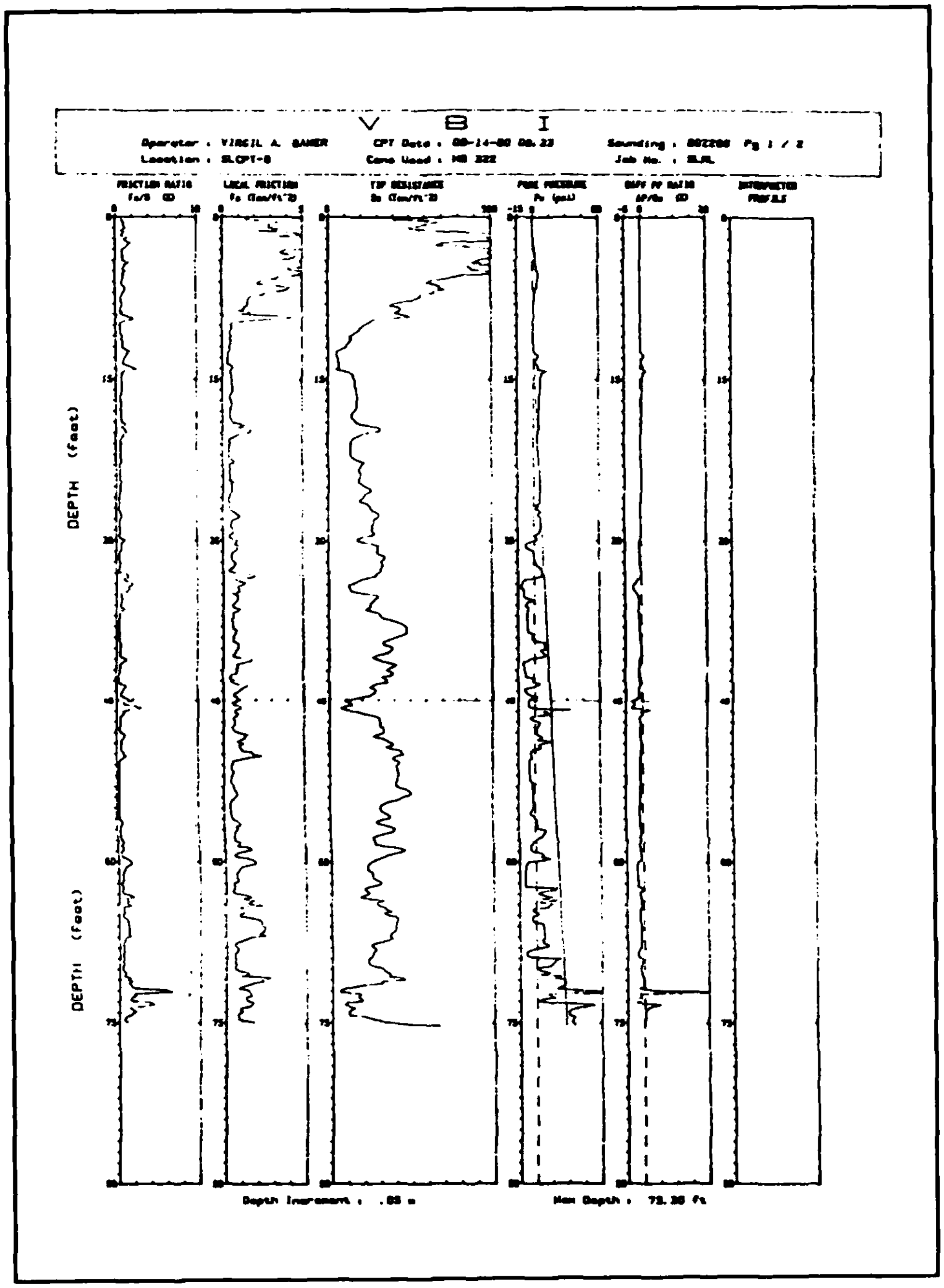


Figure 5. Sample CPT plot San Lorenzo River levee (see Figure 2 for location)

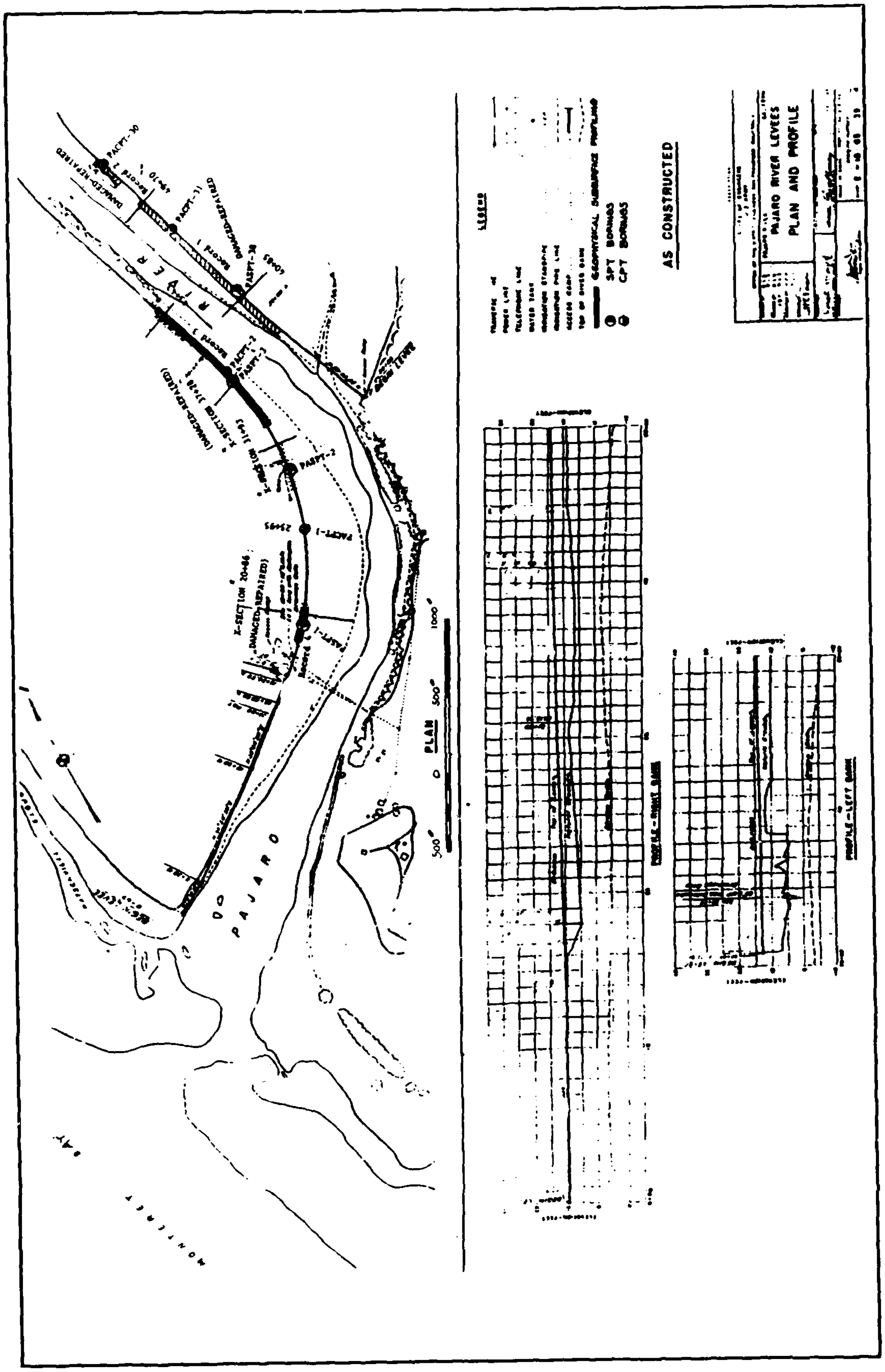


Figure 6. Location map for Pajaro River levee

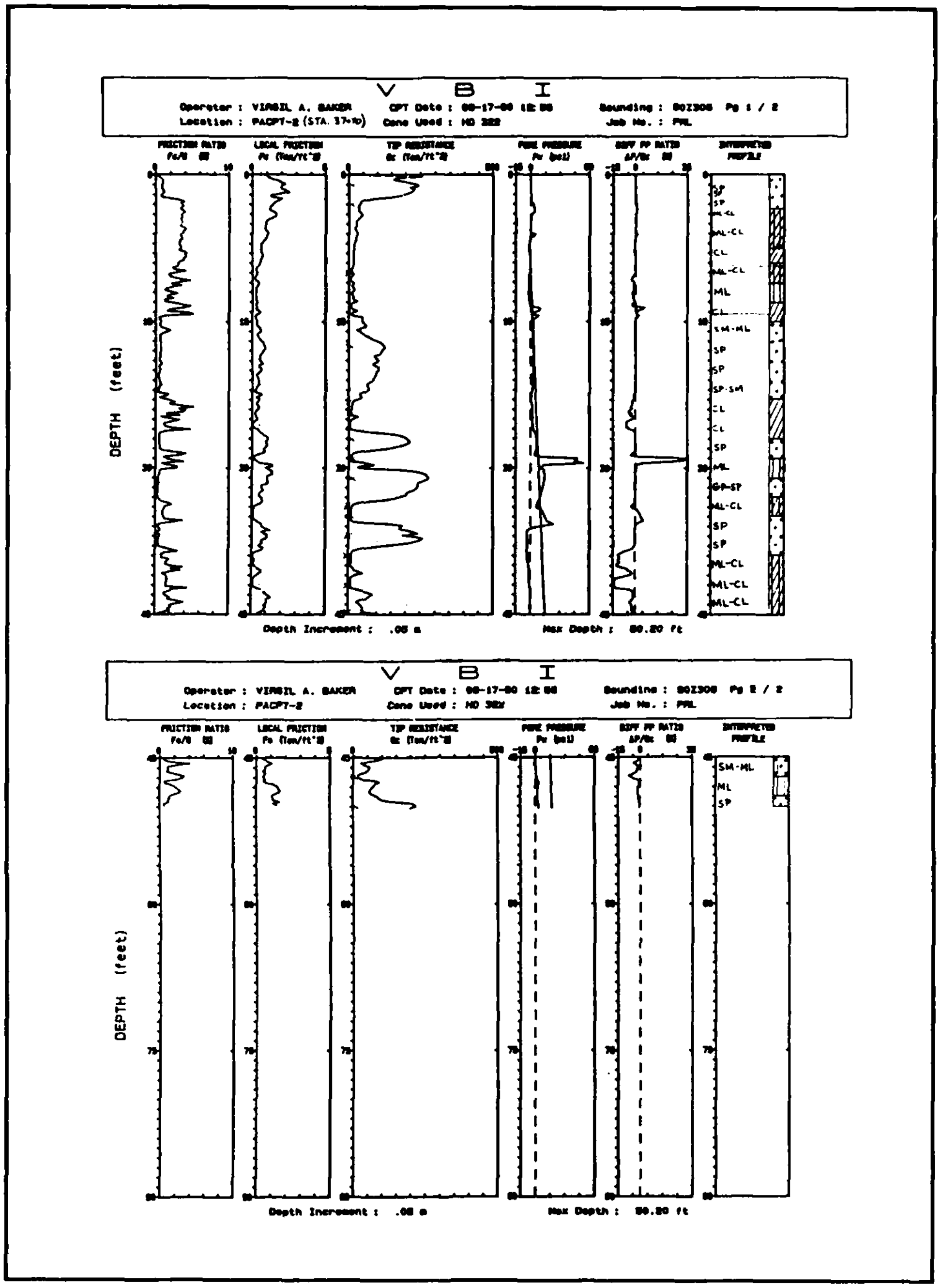


Figure 7. Sample CPT plot, Pajaro River levee (see Figure 6 for location)

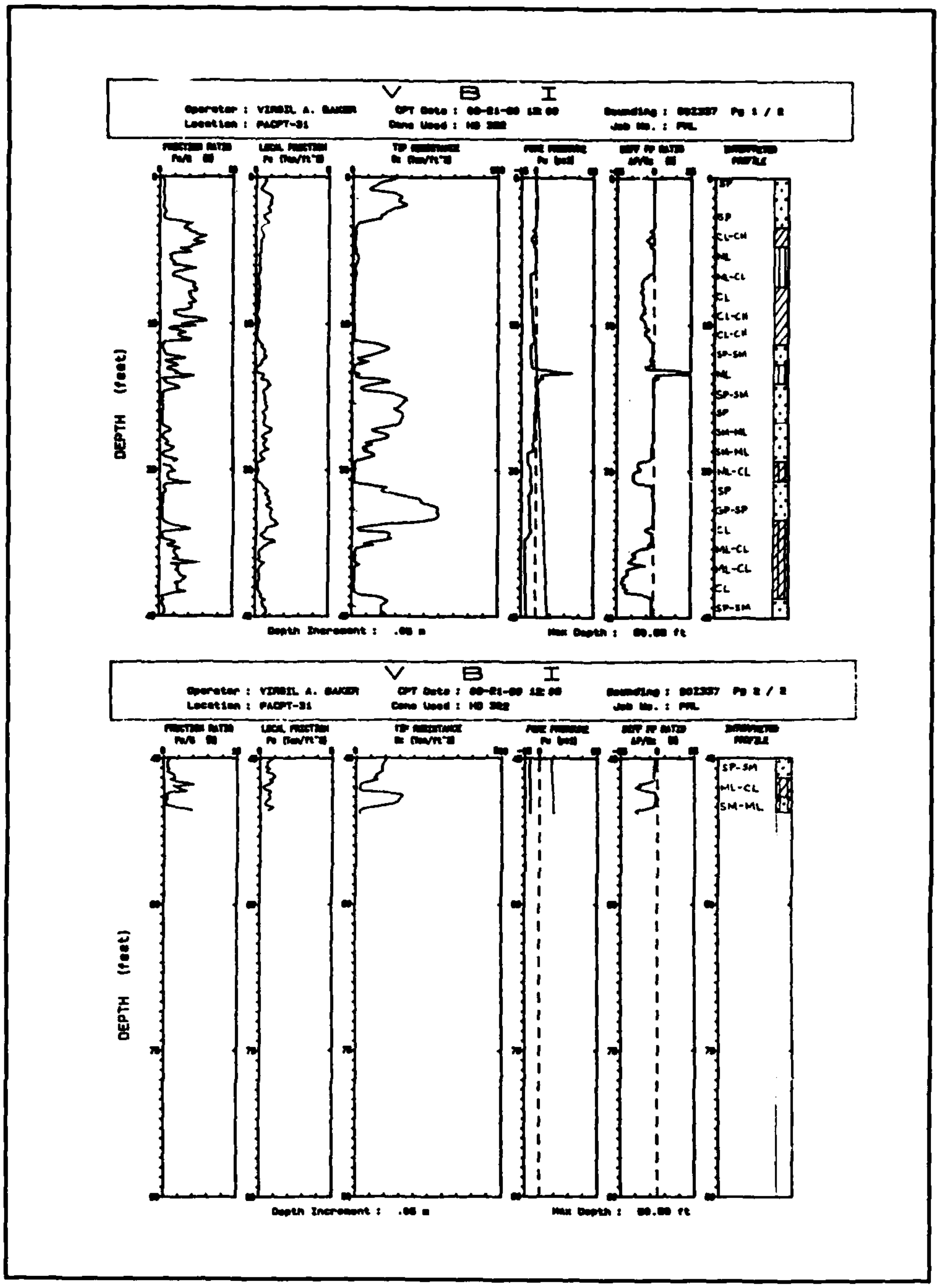


Figure 8. Sample CPT plot, Pajaro River levee (see Figure 6 for location)

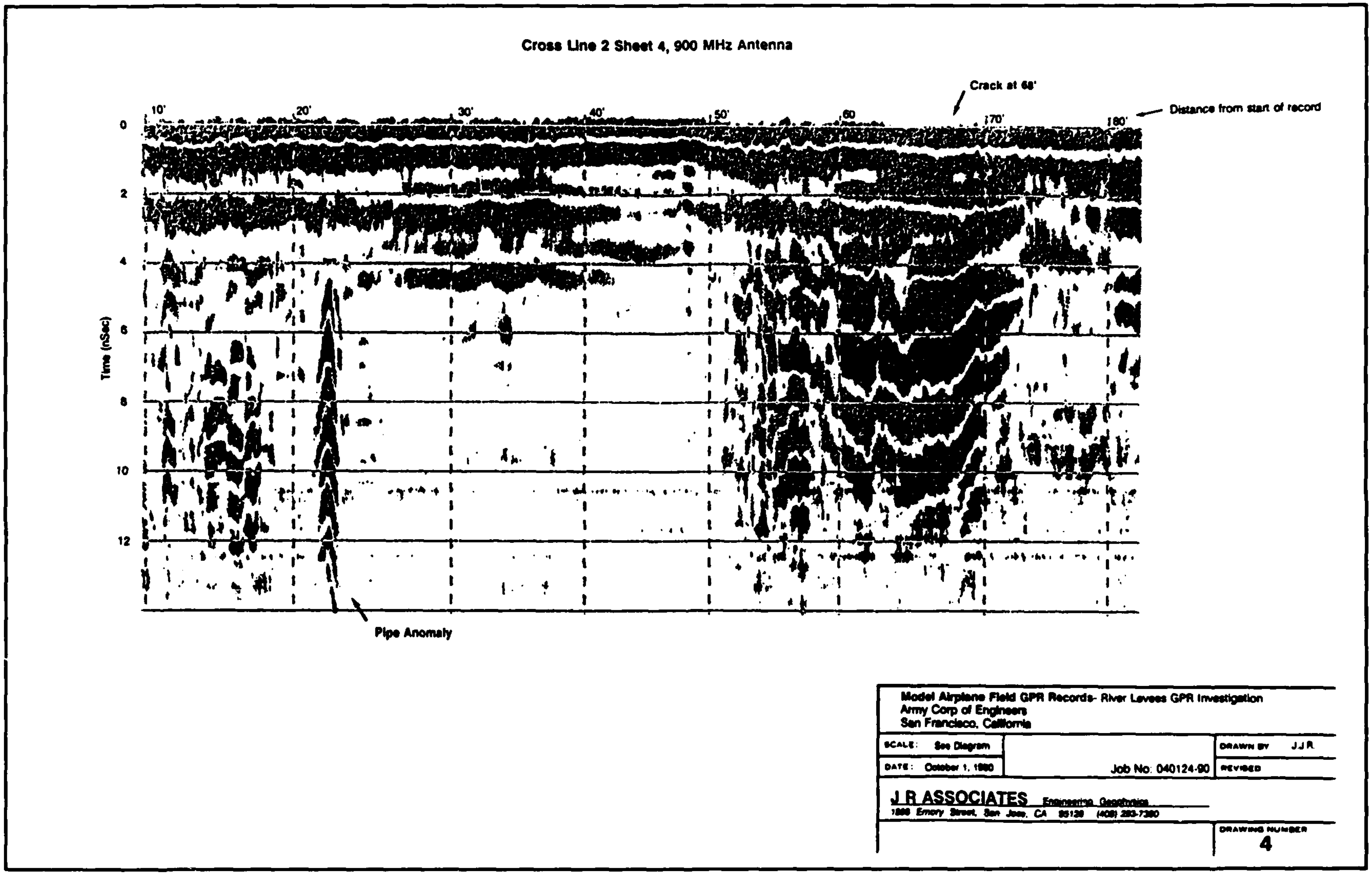
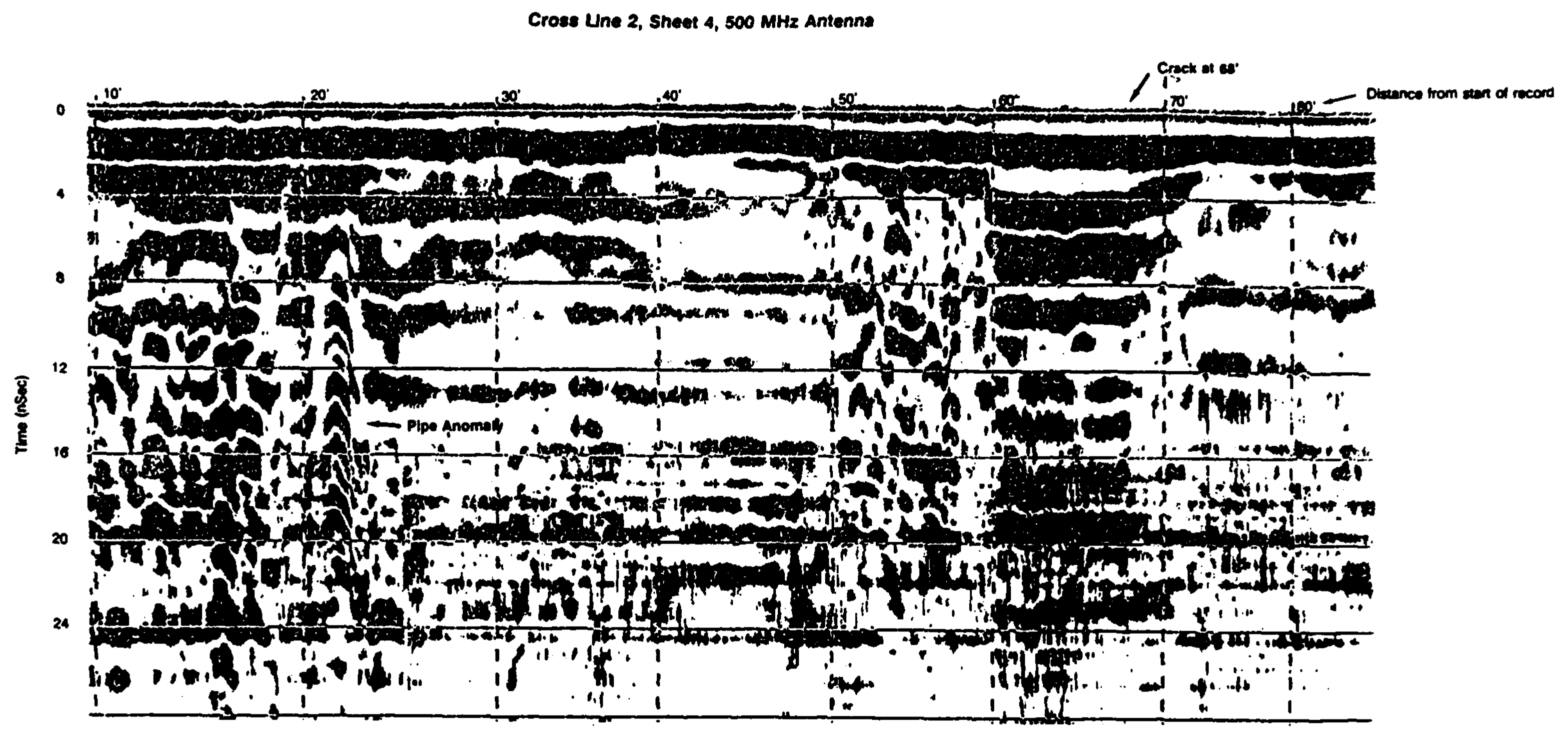


Figure 9. Pajaro River levee, sample GPR record across AC pavement and berm on riverside of levee



Model Airplane Field GPR Records, Continued- River Levees GPR Investigation
Army Corp of Engineers
San Francisco, California

SCALE: See Diagram
DATE: October 1, 1990
Job No: 040124-90
DRAWN BY J J R
REVISED

J R ASSOCIATES Engineering Geophysics
1886 Emory Street, San Jose, CA 95126 (408) 293-7390

DRAWING NUMBER
5

Figure 10. Pajaro River levee, sample GPR record across AC pavement and berm on riverside of levee

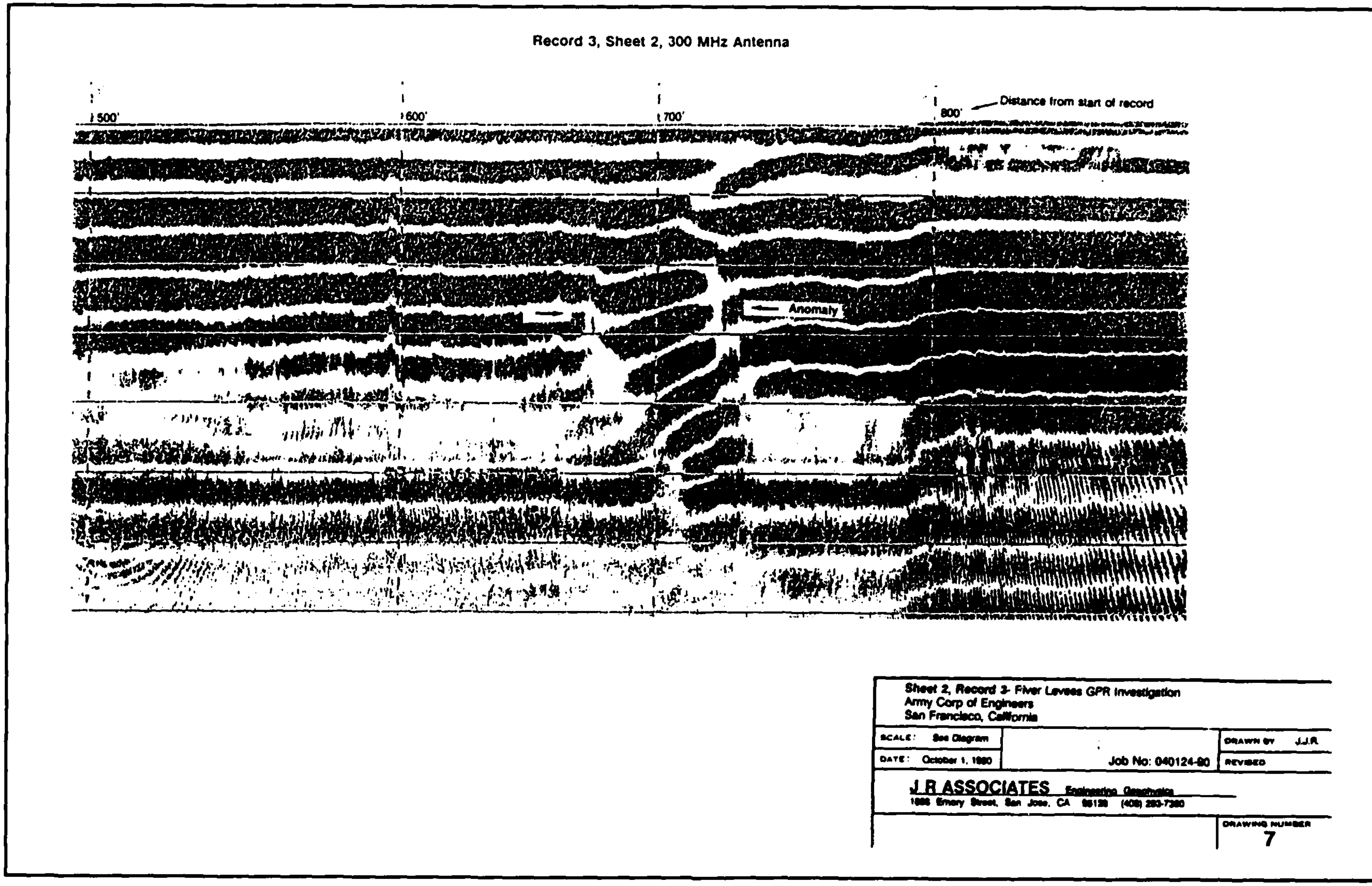


Figure 11. Pajaro River levee, sample GPR record through top of levee

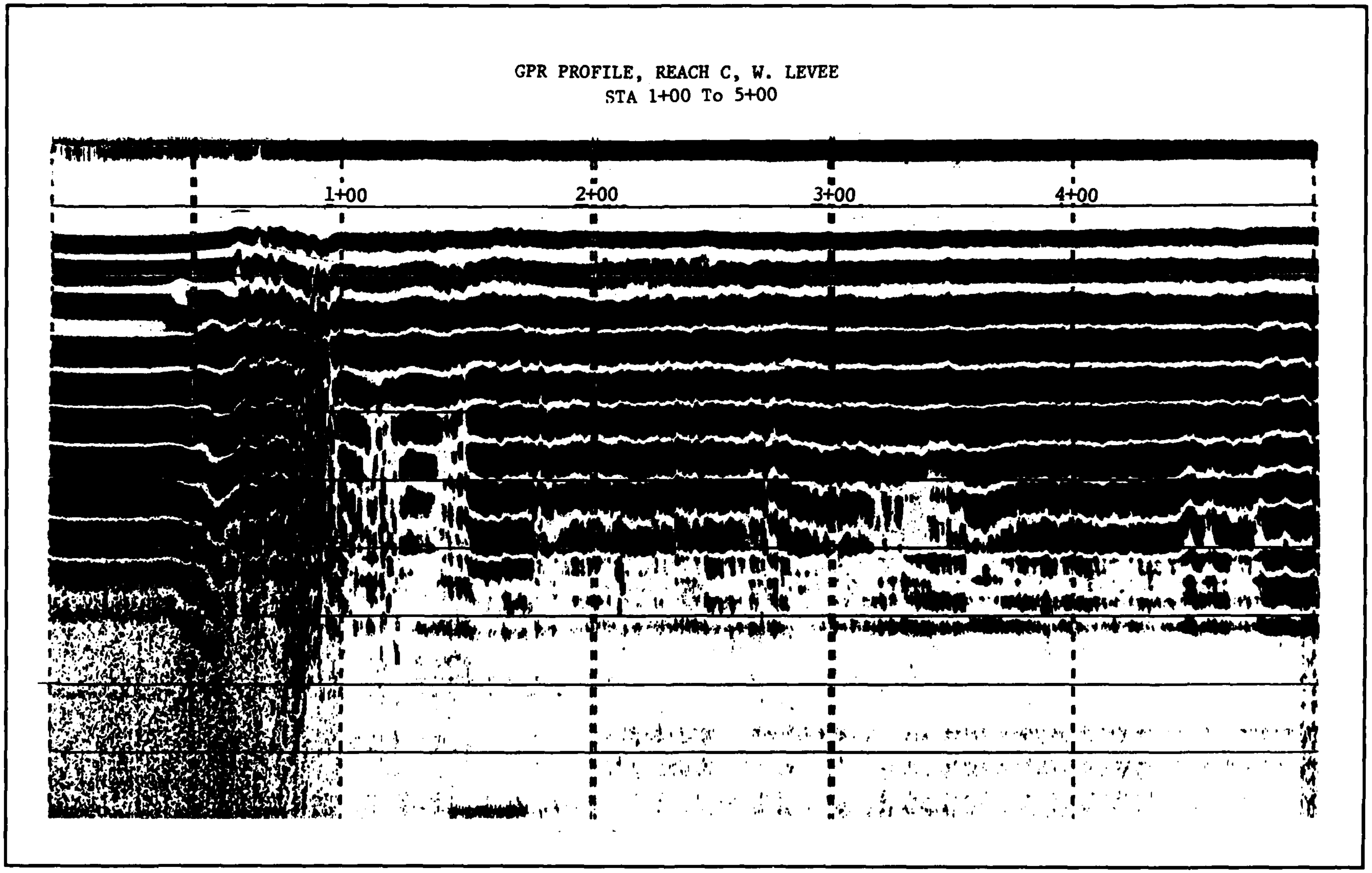


Figure 12. San Lorenzo River, sample GPR record, repaired levee, 300 Mhz, 60 nsec record

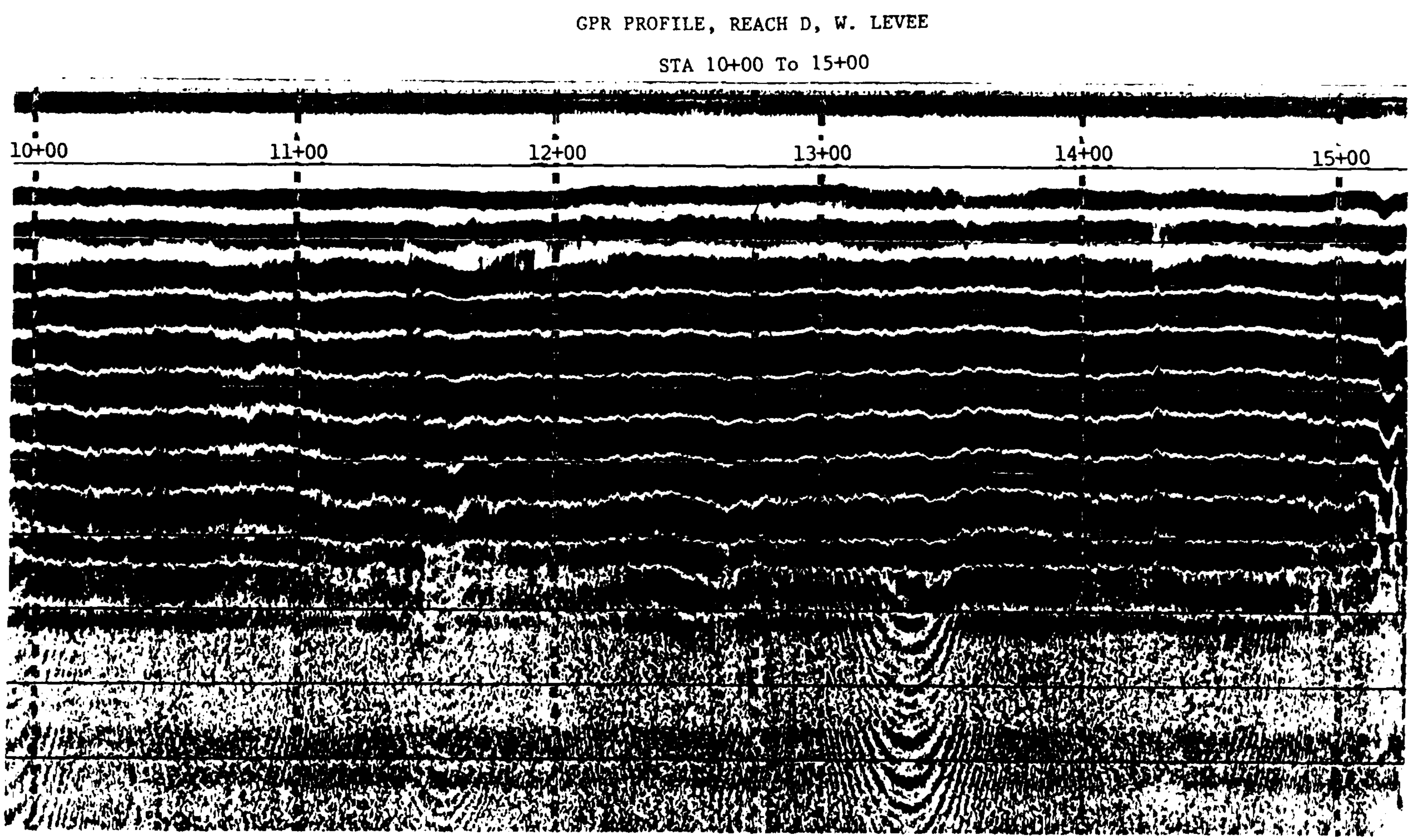


Figure 13. San Lorenzo River, sample GPR record, repaired levee, 300 Mhz

The results of the GPR records obtained along the San Lorenzo River levees are similar to those obtained along the Pajaro River levees. Approximately 18,000 lin ft of record were obtained along the entire levee system in Santa Cruz, California.

Liquefaction Assessment

The liquefaction potential of the San Lorenzo River levee foundation materials was assessed using the SPT N-values corrected to N_1-values for magnitudes 6, 6.5, and 7 earthquakes. Charts and tables by Seed et al., and the equation by Finn and Atkinson were used to compute the capacity, C, of the system in terms of cyclic stress ratio, CSR. The demand, D, on the system in terms of CSR was also computed by equation. The equations used are:

$$CSR = N_1/(12.9M - 15.7)K_sK_o = Capacity,\ C \tag{1}$$

$$CSR = CSR_{TAB} \times K_m \times K_s \times K_o = Capacity,\ C \tag{2}$$

$$CSR = 0.65\ (\sigma_v/\sigma_v')r_d \times a = Demand,\ D \tag{3}$$

where

N_1 = SPT N values correction for overburden, fines content, and rod length (No hammer energy correction was required, test performed according to ASTM Standard)

M = Earthquake magnitude

σ_v, σ_v' = Total and effective overburden stress at bottom of layer under consideration

r_d = Reduction factor (0.95 used)

a = Ground acceleration in g's

K_m, K_s, K_o = Correction factors for magnitude, slope, and confining pressure, respectively

Equation 2 is based on data presented by Seed et al. (1985) as shown in Figures 14 through 17. Equation 1 is based on the relationship presented by Finn and Atkinson (1985) as shown in Figure 18. Equation 3 is from a paper presented by Seed and Idriss (1983).

Liquefaction analyses were performed in four reaches that sustained major damage during the Loma Prieta earthquake. Equations 1, 2, and 3, and

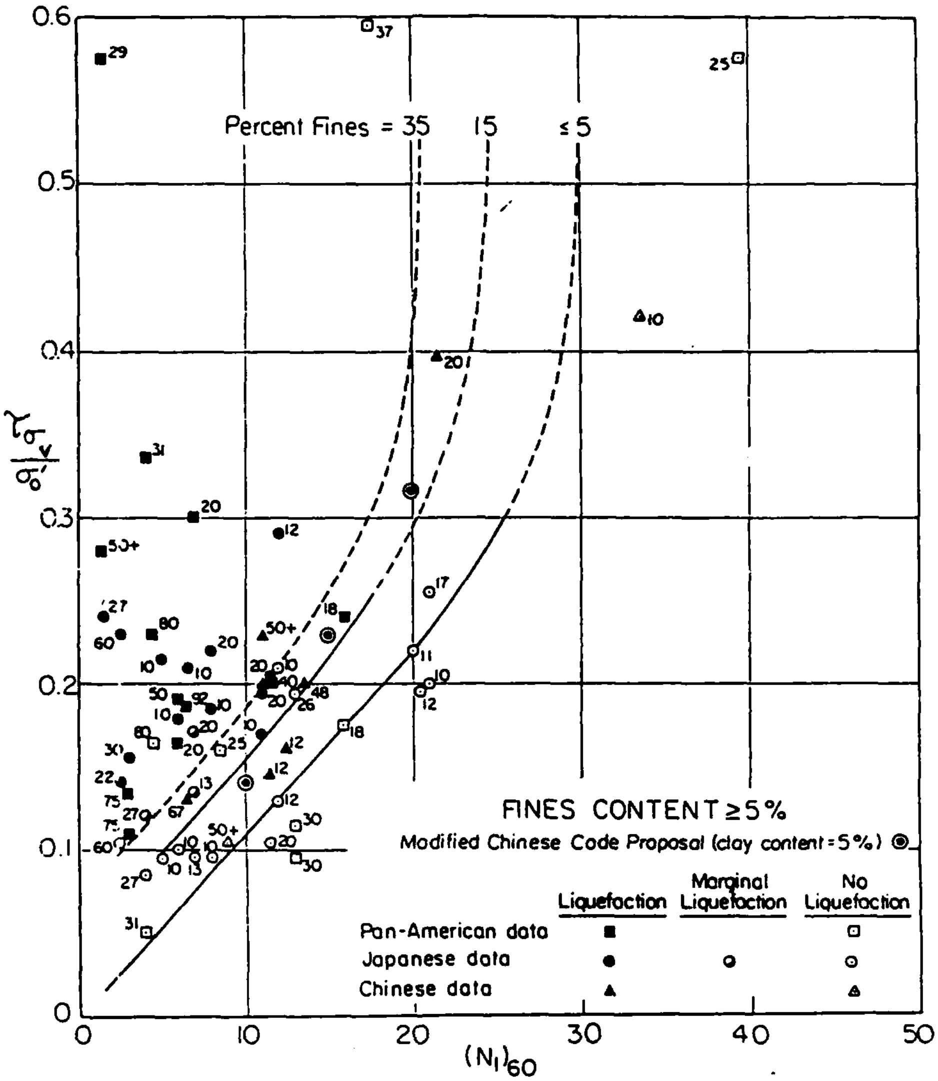


Figure 14. Relationships between stress ratio causing liquefaction and N_1-values for silty sands for M = 7-1/2 earthquakes, (Seed et al. 1984)

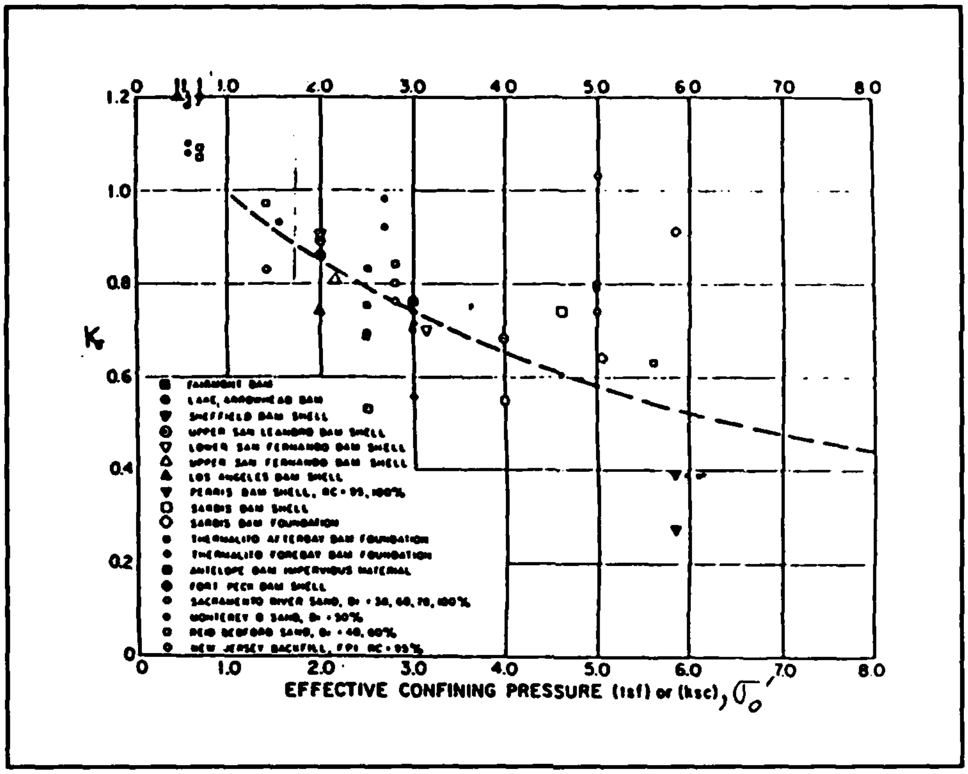


Figure 15. Relationship between effective vertical stress (σ'_o) and K_σ (Seed and Harder 1990)

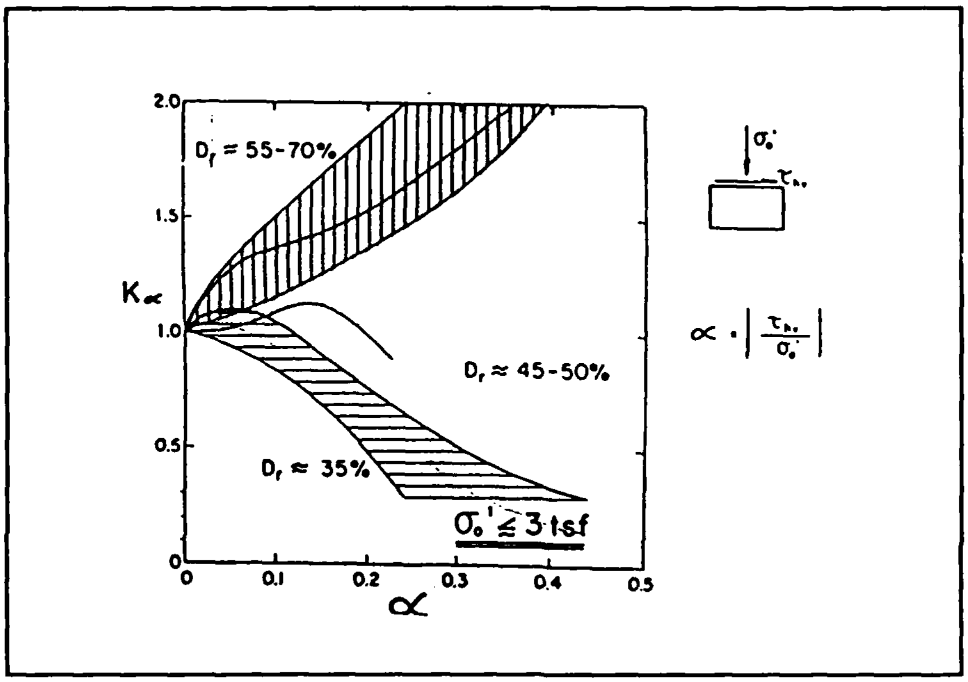


Figure 16. Relationship between α and K_α (Seed and Harder 1990)

Earthquake Magnitude, M	No. of representative cycles at 0.65 $\tau_{cyclic, max}$	Magnitude or Duration Correction Factor: C_M
8 1/2	26	0.89
7 1/2	15	1.0
6 3/4	10	1.13
6	5-6	1.32
5 1/4	2-3	1.5

Figure 17. Relationship between magnitude, number of equivalent uniform load cycles, and liquefaction resistance factor C_M (Seed and Harder 1990)

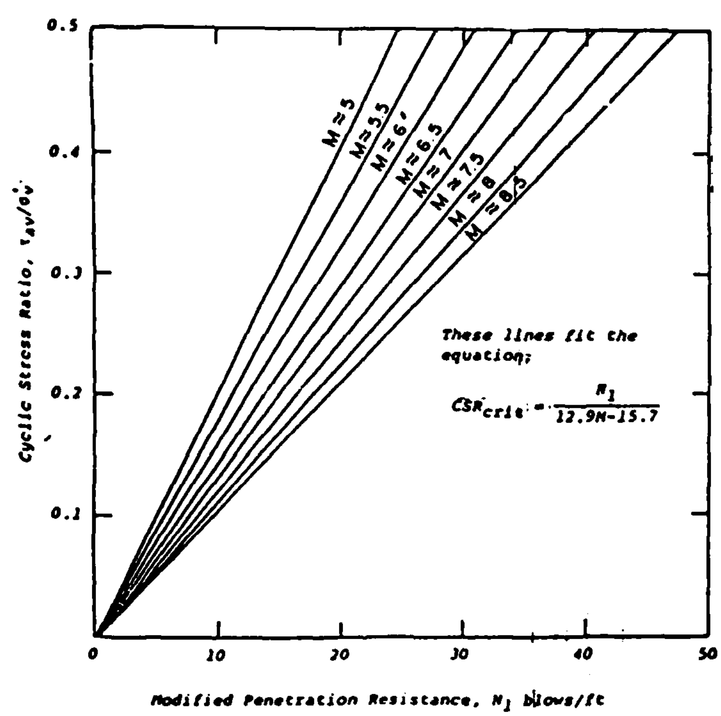


Figure 18. Mean liquefaction criteria for sands and different magnitude earthquakes (Finn and Atkinson 1985)

maximum ground accelerations of 0.36, 0.43, 0.5 g's at the site were used in the analyses. The reaches are shown in Figure 2. The typical cross section used for Reach A is shown in Figure 19. Plots showing the results of the analyses are shown in Figures 20 through 22 for Reach A. The plots show the results of the analyses for column B through the top of the levee. Data were obtained only through the top of the levee; therefore, the plotted results are based on measured field data. The accelerations used are based on a seismic assessment of the area.

Equations 1 and 3 were equated in order to make a more generalized assessment of the liquefaction potential of the San Lorenzo River and Pajaro River levees foundation materials. The correction factors for slope and confining pressures were omitted in Equation 1. By equated Equations 1 and 3, the critical acceleration, a_c, required to cause liquefaction at the project site was determined. The critical distance, R_c, from the project site to the source that causes liquefaction at the site was computed using the a_c and the acceleration attenuation equation by Idriss (1985).

$$\ln(a) = \ln \Lambda(M) - \beta(M) \ln(R+20) \pm \Sigma(M) \tag{4}$$

where a is the critical acceleration in g's, M is the surface wave magnitude for $M \geq 6$, $R = R_c$ is the closest distance in kilometers to the fault rupture point. Values of (M), β(M), and the standard error term Σ(M) are given below for the three earthquake magnitudes considered:

M	Λ(M)	β(M)	Σ(M)
6.0	282	2.07	0.42
6.5	164	1.85	0.38
7.0	91.7	1.63	0.35

One standard error term, Σ(M), was added to the attenuation equation in order to compute a_c for liquefaction. Table 3 summarizes the results of the computations for Reaches A and C. The R_c values indicate the maximum distance from the project site that an earthquake can occur and cause liquefaction at the site. Representative R_c values are plotted on the fault map shown in Figure 23 for each magnitude earthquake considered. The reaches which contain potential liquefiable materials are A, B, C, D, and E. Liquefiable layers and zones of liquefiable material were found in these reaches. Thin layers of liquefiable material were also found in reaches near the upstream end of the project. No major or significant surface damage occurred in these reaches, except for a few minor cracks in Reach J.

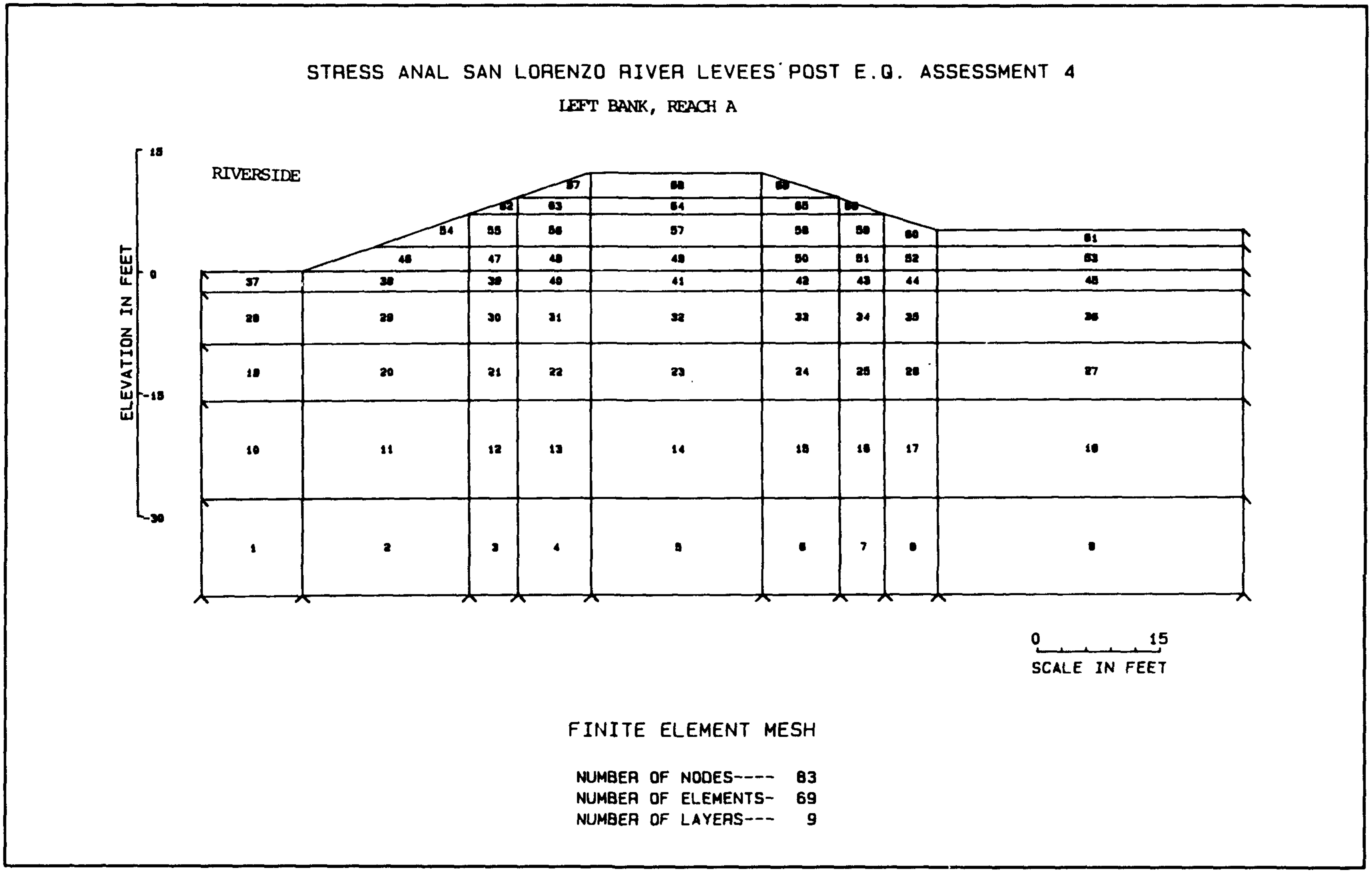


Figure 19. Stress analysis, San Lorenzo River levees post earthquake, assessment 4

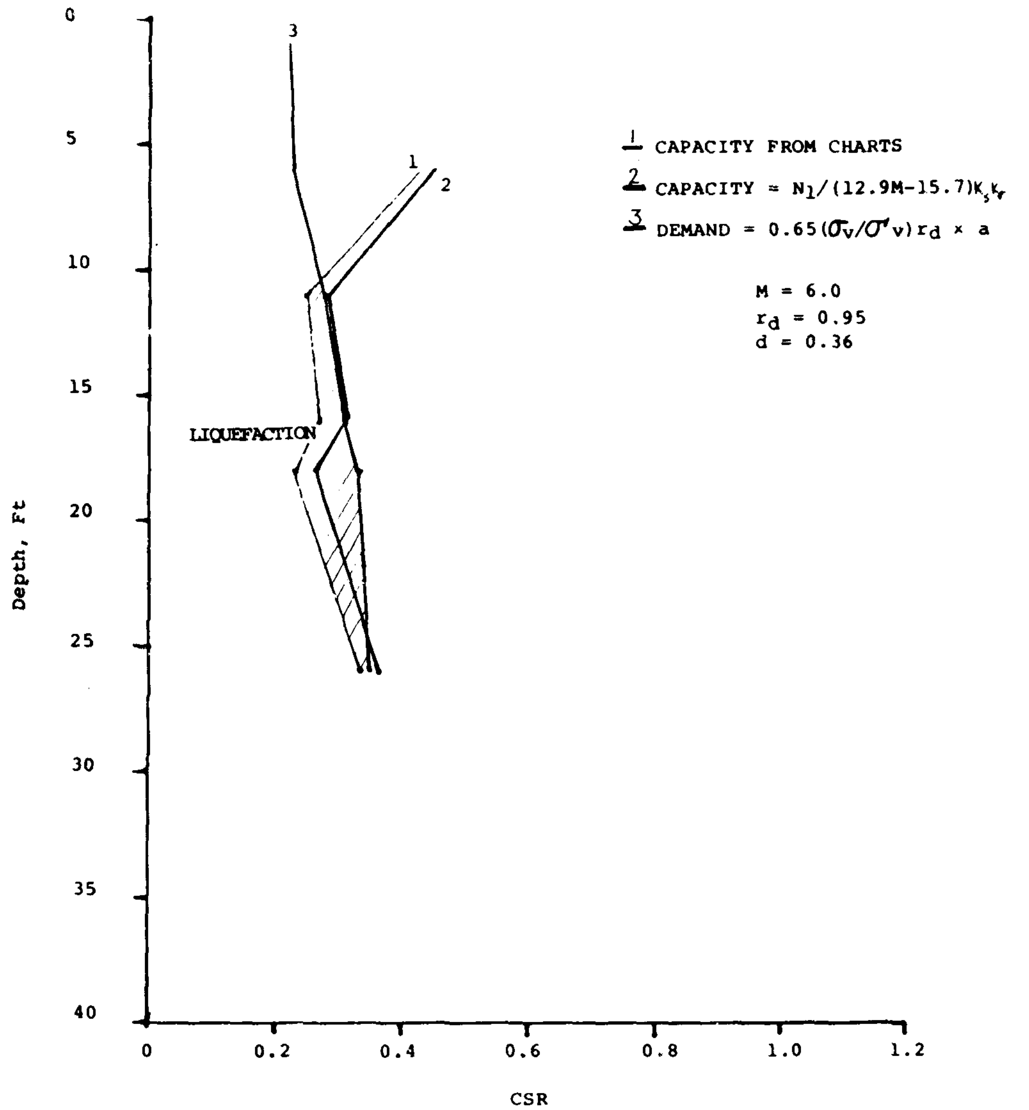


Figure 20. Liquefaction assessment, east levee, reach a, boring SL-7-90, column B

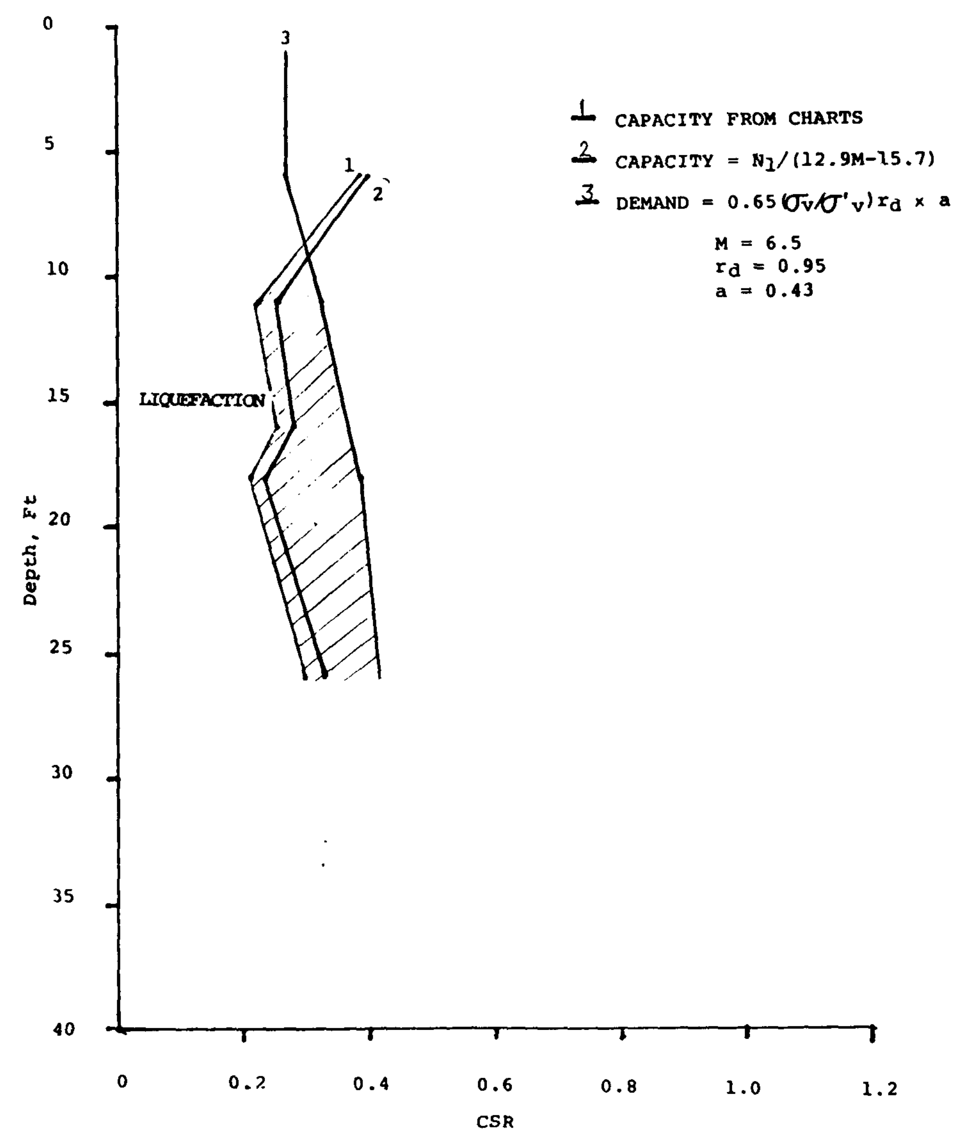


Figure 21. Liquefaction assessment, east levee, reach a, boring SL-7-90, column B

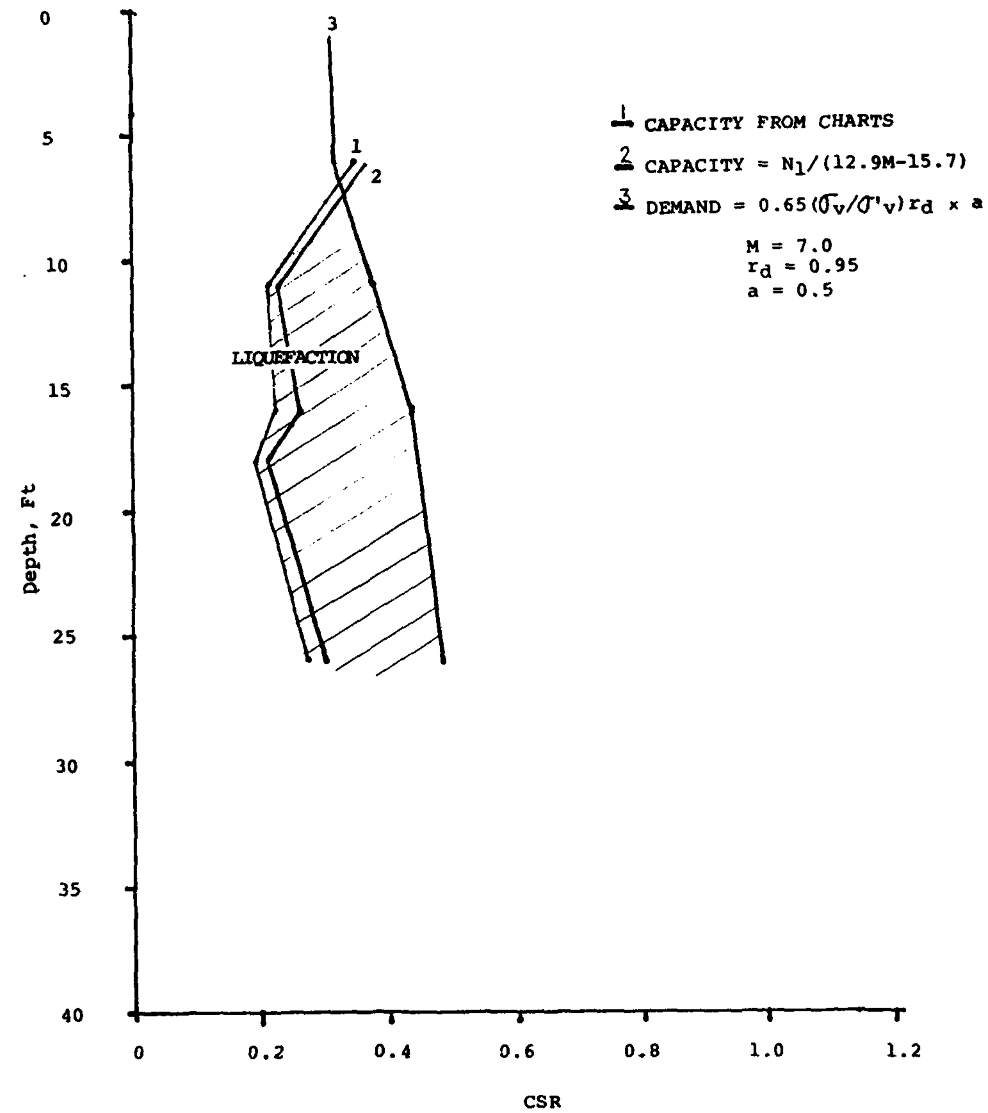


Figure 22. Liquefaction assessment, east levee, reach a, boring SL-7-90, column B

Table 3
Summary of Liquefaction Assessment

BORING No. SL-1-90   REACH C

APPROX. ELEV. ________

DEPTH (FT)	SOIL CLASS. (USCS)	N_1	M = 6.0				M = 6.5				M = 7.0				M =			
			C	D	a_c	R_c	C	D	a_c	R_c	C	D	a_c	R_c	C	D	a_c	R_c
0	SV-SM																	
5		36.5	0.592				0.536				0.489							
10	SV	29	0.470	$0.651a_c$	0.722	1.885	0.426	$0.651\ _c$	0.654	4.335	0.389	$0.651a_c$	0.597	7.198				
15	SV (5% FINES)	14	0.227	$0.776a_c$	0.292	13.868	0.205	$0.776a_c$	0.265	19.666	0.188	$0.776a_c$	0.242	27.355				
20	SV-SM (10% FINES)	19.5	0.316	$0.859a_c$	0.368	10.310	0.286	$0.859a_c$	0.333	15.034	0.261	$0.841a_c$	0.311	20.599				
25		24.5	0.397	$0.918a_c$	0.433	8.031	0.360	$0.918a_c$	0.392	12.099	0.328	$0.909a_c$	0.361	17.018				
30	SM (34% FINES)	20.5	0.332	$0.962a_c$	0.345	11.251	0.301	$0.962a_c$	0.313	16.252	0.275	$0.932a_c$	0.295	21.933				
35																		
40	SV-SM (9% FINES)	20.5	0.332	$1.024a_c$	0.324	12.208	0.301	$1.024a_c$	0.294	17.497	0.275	$0.954a_c$	0.288	22.538				
45																		

N_1 = CORRECTED SPT BLOW COUNT_CORRECTIONS FOR OVERBURDEN, ROD LENGTH, AND FINES

C = CAPACITY OR STRENGTH OF SYSTEM $\cong N_1/(12.9M \cdot 15.7)$, CSR

D = DEMAND ON SYSTEM = $0.65\ (\sigma_v/\sigma_v')\ r_d \times a$

a_c = CRITICAL ACCELERATION FOR LIQUEFACTION AT THE SITE

R_c = CRITICAL DISTANCE WITHIN WHICH AN EARTHQUAKE MUST OCCUR TO CAUSE LIQUEFACTION

SEE FIGURE 4.1 FOR SPT BORING LOCATIONS

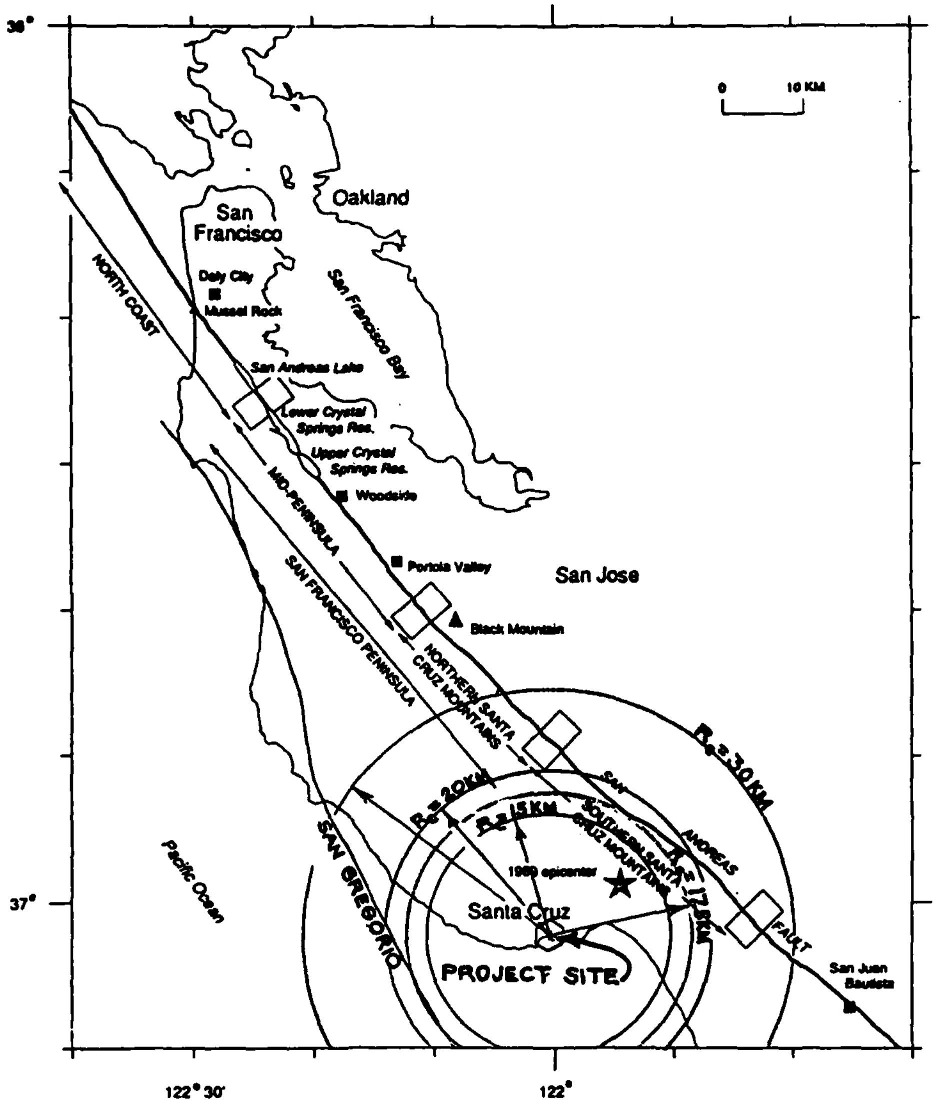
(Continued)

Table 3 (Concluded)

BORING No. SL-7-90 REACH A

APPROX. ELEV. ________

DEPTH (FT)	SOIL CLASS. (USCS)	N_1	M = 6.0				M = 6.5				M = 7.0				M =			
			C	D	a_c	R_c	C	D	a_c	R_c	C	D	a_c	R_c	C	D	a_c	R_c
0	SW																	
5	SM	68.5	1.110				1.005				0.918							
10 (▽)		25.5	0.413	$0.651a_c$	0.635	3.288	0.374	$0.651a_c$	0.575	6.087	0.342	$0.651a_c$	0.525	9.431				
15	SM (18% FINES)	16.5	0.267	$0.776a_c$	0.345	11.284	0.242	$0.776a_c$	0.312	16.295	0.221	$0.776a_c$	0.285	22.814				
	SW-SM (7% FINES)																	
20	SW-SM (7% FINES)	18.5	0.300	$0.860a_c$	0.345	11.109	0.271	$0.860a_c$	0.316	16.068	0.248	$0.860a_c$	0.288	22.510				
	SP-SM	15.5	0.251	$0.885a_c$	0.284	14.357	0.227	$0.885a_c$	0.257	20.307	0.208	$0.885a_c$	0.235	28.224				
25																		
30	SP-SM (12% FINES)	22.5	0.365	$0.962a_c$	0.379	9.876	0.330	$0.962a_c$	0.343	14.473	0.302	$0.962a_c$	0.314	20.383				
35																		
40	COHESIVE LENSE	6	0.097				0.088				0.080							
	ML (NON-LIQ.)	35	0.567	$1.033a_c$	0.549	4.979	0.514	$1.033a_c$	0.497	8.215	0.469	$1.033a_c$	0.454	12.170				
45																		



NOTE: San Andreas fault segments. Stippled rectangles are segment boundary zones. Star marks epicenter of 1989 Loma Prieta earthquake.

R_c=CRITICAL RADIUS FOR LIQUEFACTION AT THE PROJECT SITE. AN EARTHQUAKE OCCURRING BEYOND R_c CAUSES NO LIQUEFACTION AT THE SITE

R_c=30KM CORRESPONDS TO 7.0 M EARTHQUAKE

R_c=20KM CORRESPONDS TO 6.5 M EARTHQUAKE

R_c=15KM CORRESPONDS TO 6.0 M EARTHQUAKE

Figure 23. Location of project site with respect to 1989 Loma Prieta earthquake

Foundation Strengthening, San Lorenzo River Levees

Reaches

Liquefiable foundation materials could cause damage to the levees in future earthquakes in the following reaches:

a. Reach A, west levee, approximately 1,800 lin ft.

b. Reach B, west levee, approximately 500 lin ft.

c. Reach C-D, east levee, approximately 1,500 lin ft.

d. Reach E, east levee, approximately 500 lin ft.

The liquefiable materials within the reaches could be strengthened using a method that does not require extensive degrading of the existing levee embankment or significantly changes and restricts groundwater movement within the foundation. The following list includes possible ground modification methods for strengthening liquefiable materials:

a. Dynamic deep compaction.

b. Vibro-compaction.

c. Compaction grouting.

d. Chemical grouting.

e. Jet grouting.

f. Vibro-replacement (Stone columns).

Compaction grouting and vibro-compaction appear to be two methods suitable for strengthening the liquefiable layers with minimum impact to the existing levee embankment and groundwater movement. These two methods are discussed below.

Compaction (Displacement) Grouting

A highly viscous soil, cement, and water, pressurized grout is used to radially compress or compact the surrounding soil. Grout pipes are installed and compaction grouting done a predetermined pattern and depth. Figure 24 shows a conceptual design of this scheme.

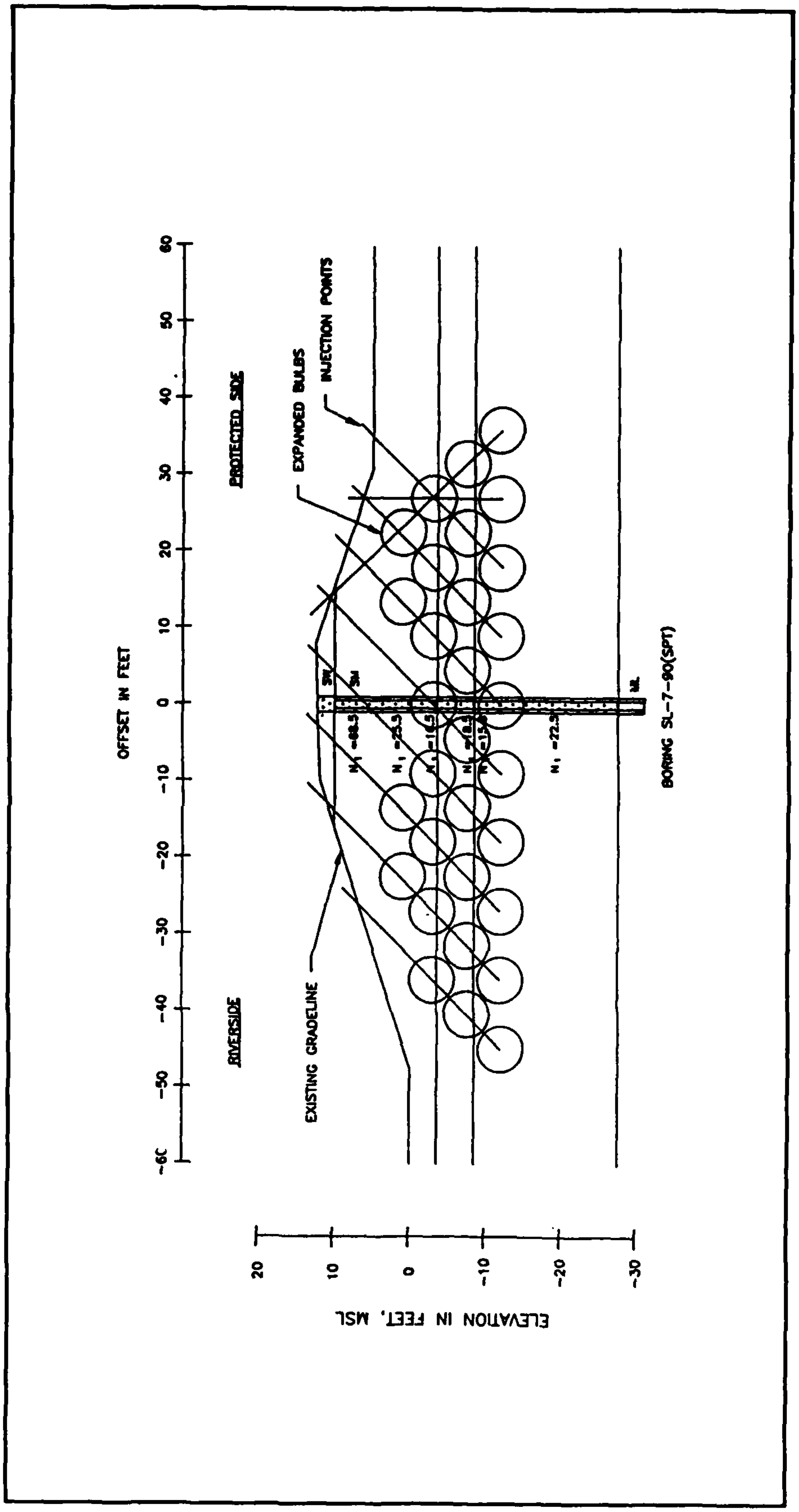


Figure 24. Compaction grouting conceptual design

Vibro-Compaction

This method involves the rearrangement of soil particles into a denser configuration by use of powerful vibrators. The probes are inserted into the ground to the bottom of the soil layer which is to be densified. The probes are inserted and vibration performed in a predetermined pattern. A strong vibration is applied as the probes are slowly withdrawn. Preliminary analysis indicate that the probes should be inserted and vibration performed on a 6-ft center to center grid spacing. A conceptual design is shown in Figure 25.

Seismic Analysis

A comprehensive seismic analysis was performed for the San Lorenzo River levees. The seismicity of the area was determined. A combined probability of liquefaction was computed based on all the active faults that are capable of causing liquefaction within the area. A risk analysis was performed to determine the hazard and economic justification for taking any seismic corrective measures to strengthen the liquefiable foundation materials. Details of the seismic analyses were included in the engineering appendix ± prepared for the San Lorenzo River flood control feasibility study which is currently under review by higher authority, headquarters.

Conclusions

San Lorenzo River Levees

Liquefiable foundation materials were found throughout the project site. There is a reasonable chance of an earthquake occurring on the San Francisco Peninsula segment of the San Andreas fault and causing liquefaction at the site. There are also other faults that have a high potential of causing liquefaction at the site. These faults include the Zayante-Vergeles Fault, the San Gregorio-Seal Cove Fault and the Monterey Bay fault zone. The Monterey Bay fault zone and the Zayante-Vergeles fault are capable of earthquakes with a return period of approximately 18 years. Magnitude 6 earthquakes on these faults can cause liquefaction at the project site and possible damage to the levees. The Zayante-Vergeles fault and the San Gregorio-Seal Cove fault are capable of magnitude 6.5 earthquakes with a return period of approximately 43 to 53 years. These earthquakes can liquefy the site and possibly damage the levees. The liquefiable foundation materials need to be strengthened in order to prevent future damage to the levee and structures from liquefaction.

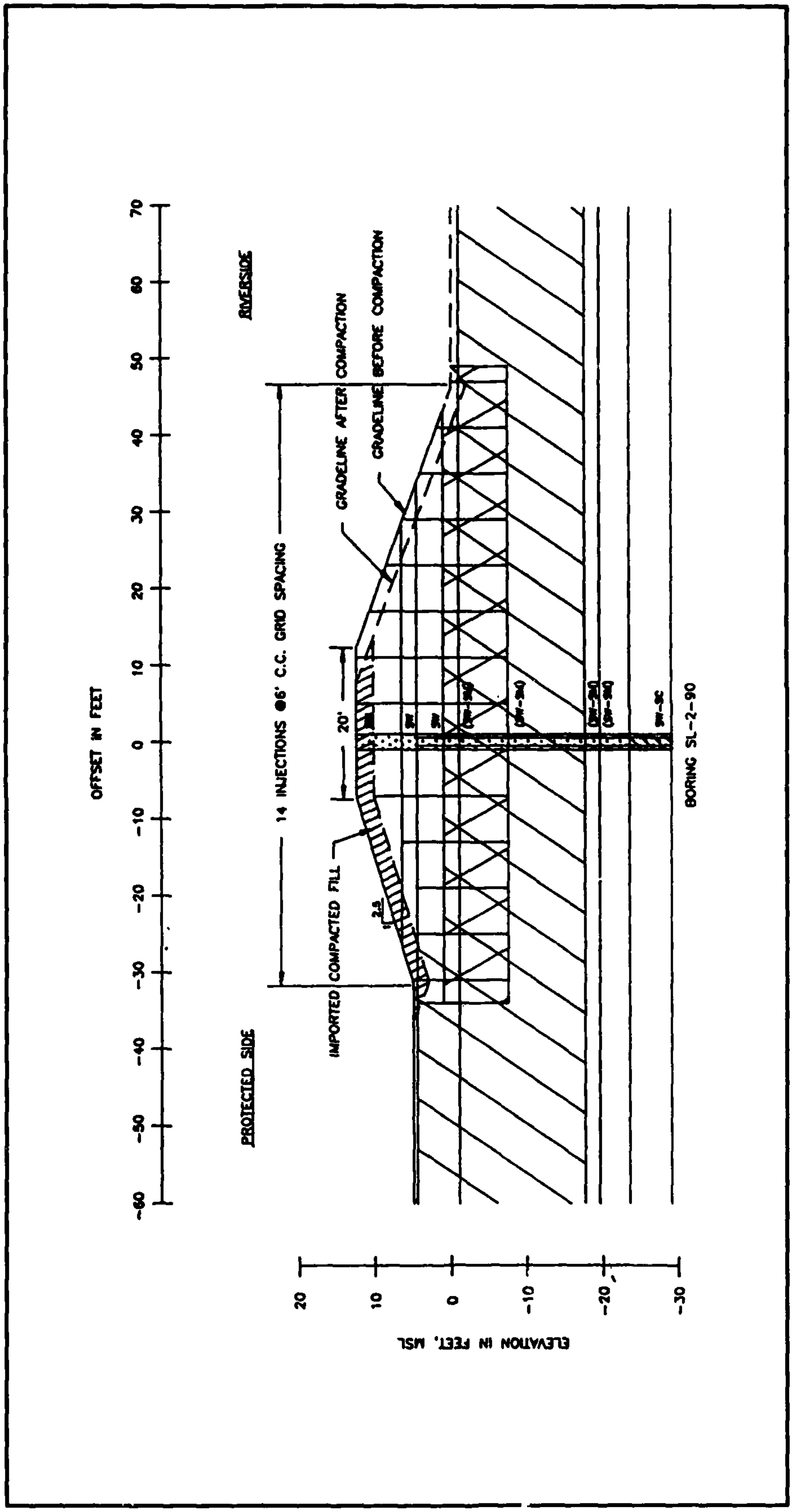


Figure 25. Vibrocompaction conceptual design

Pajaro River Levees

Liquefiable foundation materials exist throughout the levee system. The Loma Prieta Earthquake revealed most of the liquefiable sites. Other sites were revealed as a result of the geotechnical assessment. There were sites that appear to have sustained earthquake induced lateral movement and surface cracking. The materials consist of soft to very soft layers of nonliquefiable clayey soils in most cases. One of these sites is located within the City of Watsonville. Analyses of this site and two other sites in lower reaches indicate that the levee sections have an adequate static safety factor. However, large cracks appeared and was investigated during April 1991 in the repaired reach at Watsonville and adjacent to a repaired reach near Station 124+00 on the Santa Cruz County side of the river were investigated documented during April 1991.

The lower stretch of levees sustained major damage that included relatively large reaches. These reaches are within the coastal and tidal zone where the water table is generally near the ground surface.

The damaged levee reaches were repaired by excavating and recompacting the embankment materials. The materials were compacted to a very dense brittle state. This material is likely to sustain more severe cracking during another damaging earthquake because of the weak foundation materials and the brittleness of the embankment materials.

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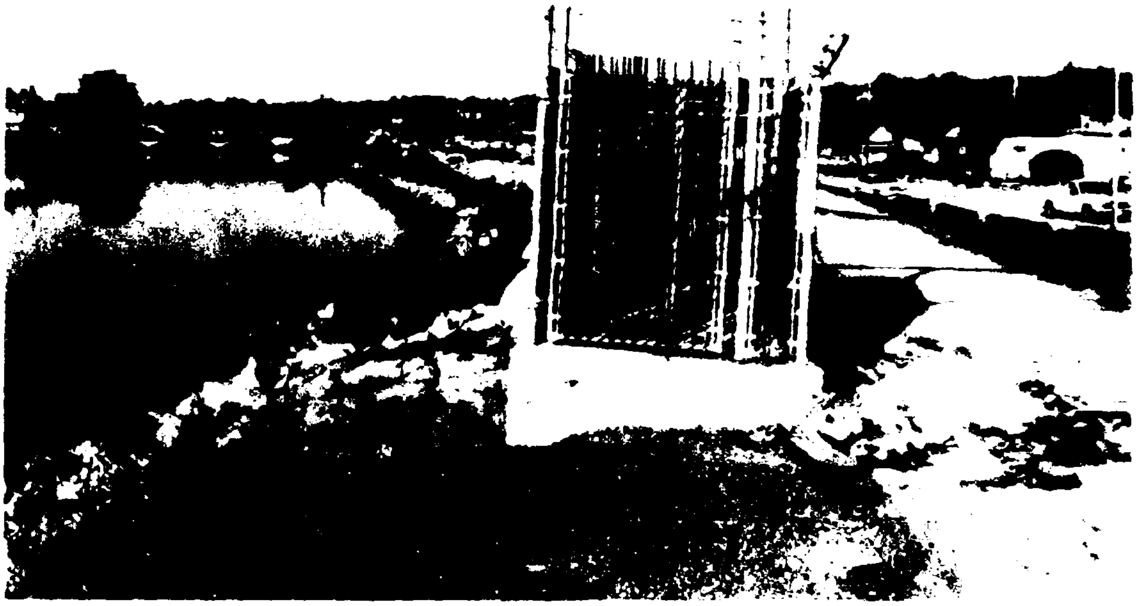
Exhibit A
Photos of Damage and Repair San Lorenzo River Levees and Pajaro River Levees

a. Closeup of cracks along crest of levee

b. Damage along levee on right bank

Photo 1. San Lorenzo River levee, Loma Prieta earthquake damage



a. Damage at drainage structure, left bank levee (looking upstream)

b. Damage to levee on left bank, San Lorenzo River, near mouth

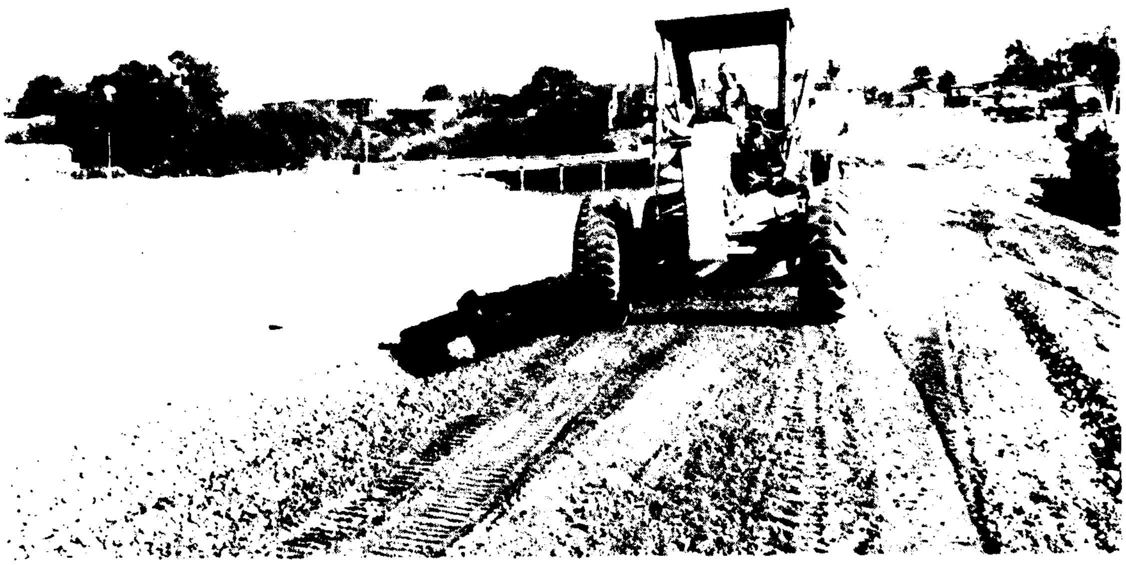
Photo 2. San Lorenzo River, Loma Prieta earthquake damage

a. Cracks along crest

b. Crack along levee slope

Photo 3. Typical damage to levees along Pajaro River

a. Water truck, watering slope during levee reconstruction



b. Grading during reconstruction of levee

Photo 4. San Lorenzo River levee, post earthquake repair

a. Levee reconstruction, protected side slope, right bank

b. Levee reconstruction patch work, uppermost reach, left bank

Photo 5. San Lorenzo River, post earthquake repair

a. Reconstructed levee with riprap in-place on riverside, lower reach

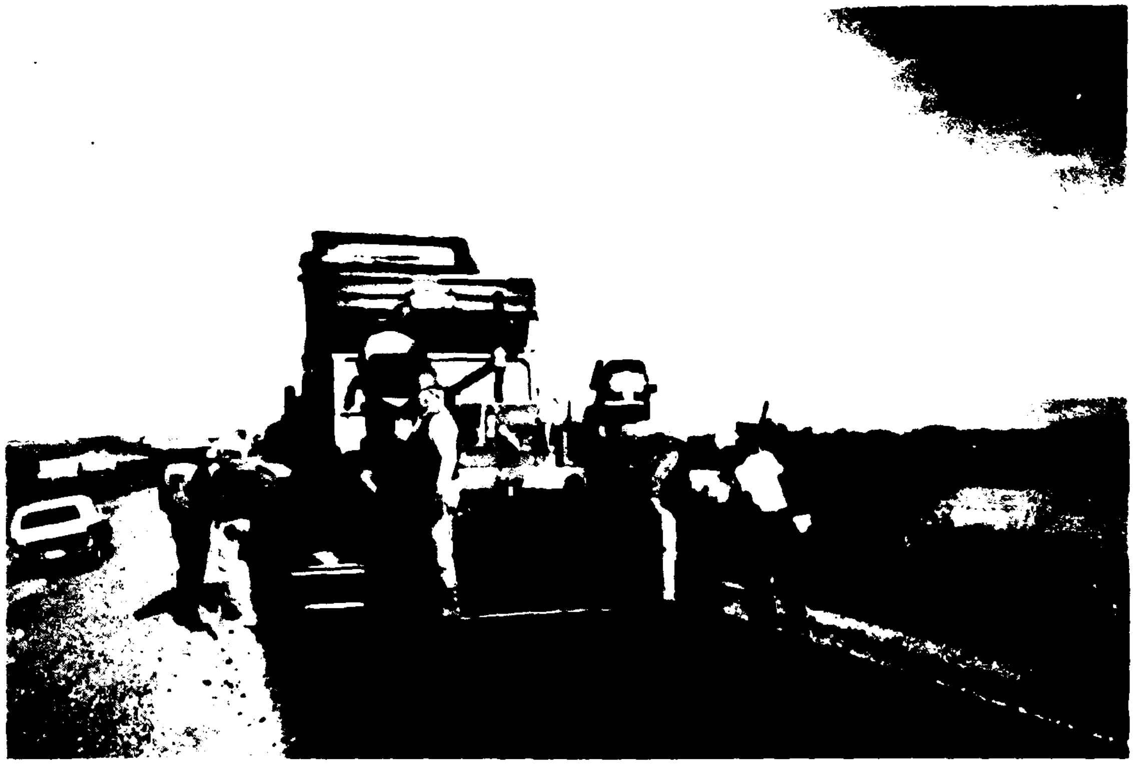
b. In-place riprap, left bank, near mouth

Photo 6. San Lorenzo River, post earthquake repair

a. Levee reconstruction on San Lorenzo River

b. Typical repair and reconstruction at drainage structures

Photo 7. San Lorenzo River levee, post earthquake repair



a. New AC pavement being placed on reconstructed levee

b. Repaired section with straw placed on protected side slope for erosion protection

Photo 8. Levee reconstruction on Pajaro River

Lime Stabilization Slide Repair at Bardwell Lake Embankment

Douglas Massoth and Bob Ehlman
Fort Worth District

Introduction

This report has been prepared for the REMR-II Workshop conducted on 17 March 1992 at Waterways Experiment Station. The Workshop was entitled "Levee Rehabilitation."

Lime stabilization of high plasticity soils to obtain better engineered fill has long been practiced throughout the world. The practice is extremely common in many parts of Texas where high plasticity, highly expansive clays occur. Lime stabilization is locally practiced for two principal reasons. First, lime stabilization is used to reduce the expansion potential of high plasticity clays under roads and structures. Second, lime stabilization is used to increase the strength gain of these soils where used in embankment fill applications. Not considering other soil stabilization methods, lime stabilization has proved to be an economical and successful method to obtain desired results.

Embankment

Bardwell Dam is located at river mile 5.0 on Waxahachie Creek in Ellis County, about 30 miles south of Dallas, Texas. The project includes an earth embankment, an uncontrolled broadcrested weir spillway, and a gated outlet works. Construction of the dam began in August 1963, and deliberate impoundment began in November 1965.

The embankment contains a volume of about 3,549,000 cu yd and is a homogeneous compacted impervious fill constructed of CL and CH clays and weathered shale. The upstream and downstream berms were constructed of compacted random fill. Unweathered shale was placed in a semi-compacted condition at the end of each berm. An inclined and horizontal drainage blanket was placed between station 87+00 and station 91+00. The upstream slope of the embankment above elevation 415.0 is protected with 18 in. of

riprap on 6 in. of bedding. Borrow material for the embankment was obtained from three upstream borrow areas and from required spillway excavation. The material from the borrow areas consisted of about 72 percent CH and 28 percent CL type clays. The compacted impervious fill zone was placed in 8-in. loose lifts and compacted by eight passes of a tamping roller. The moisture content after compaction was specified to range within the limits of optimum moisture content to 3 percentage points above optimum.

The embankment is underlain by about 40 feet of overburden over shale. Generally, the overburden consists of a dark gray clay underlain by a light gray to tan or brown clay which becomes increasingly sandy and gravelly near the top of shale. From about station 102 to station 112 the upper 8 to 15 ft of the overburden is a high plasticity clay with high moisture content and lower shear strength than in the remainder of the floodplain.

Slide History

Cracking on the downstream edge of the crest was first reported in early 1968. Initially, the cracks were filled with lean grout, later with asphalt, and then with sand. At that time it was realized that filling the cracks with sand was the worst possible treatment since this allowed a buildup of hydrostatic forces in the crack and further enhanced the potential for sliding. Subsequently, surface cracking on the crest has been a continual problem at Bardwell Dam. Since the first slides occurred in 1973, a total of 24 slides have occurred at this project. See Table 1 for a listing of all slides in the projects history.

Discovery of 1991/1992 Slides

Four shallow surface slides were discovered by Bardwell Lake personnel on 21 December 1991. Approximately 5.3 in. of rainfall was recorded for 5 days prior to the slides. The lake elevation was 427.73 ft National Geodetic Vertical Datum (NGVD) on 21 December 1991. The slides occurred between stations 79+00 to 80+00, stations 81+30 to 82+85, stations 85+50 to 87+05, and stations 101+00 to 102+35 on the downstream crest and slope of the main embankment. Project personnel documented, assessed structural stability of the embankment, and reported the damages to the appropriate CESWF Operations and Geotechnical personnel on 21 December 1991.

Three cracks were discovered by project personnel at 0900 hr on 9 March 1992 on the downstream crest of the main embankment. Approximately 5.86 in. of rainfall was recorded for a 2 week period prior to the discovery of the cracks. The lake elevation was 429.84 ft NGVD on 9 March 1992. The cracks occurred between stations 90+00 to 91+40, stations 93+00 to 94+00, and stations 95+50 to 97+50 on the downstream crest and slope. Project personnel notified CESWF Operations and Geotechnical personnel on

9 March 1992. These cracks became definable slides following a week long rainy period of 22-29 June 1992, during which time 2.8 in. of rainfall was recorded. During and following another rainy period from 20 July to 3 August 1992, when 4.64 in. of rainfall was recorded, significant movement of slide debris was observed at these locations. The lake elevation was 421.75 ft NGVD on 4 August 1992.

Description of 1991/1992 Slides

The following is a description and location of the seven slide areas. Some of the descriptions were made immediately prior to the start of repairs and some of the descriptions and findings were made as the slide debris was excavated.

Station 79+00 to Station 80+00 This slide did not display any early warning features such as cracking or bulging before failure and displacement of material occurred. Four electric service power poles are located between stations 74+70 and 81+00 along the downstream crest to provide aerial electric service to the Outlet Works structures. These poles have not been plumb and have been leaning in the downstream direction for many years. These poles, wires, transformer banks, meter pole, siren pole, and switch panels had to be removed before repairs began. The Bardwell Project Office portable generator supplied the temporary electrical power to the Outlet Works structure until permanent electric service could be reinstalled. This slide, before repairs began on 10 August 1992, had a 12-ft exposed vertical scarp about 8 ft into the downstream crest, parallel to the centerline of the embankment. The scarp was arced and extended 60 ft down the slope from the crest edge. The scarp was 100 ft long at the crest and widened going down the slope to about 150 ft long at its lower horizontal slickensided failure plane at approximate elevation 435 ft NGVD on the 1V:2.5H slope. The slide debris rolled out and over the grass surface of the 1V:6H slope, stopping at elevation 420 ft NGVD. The slide debris and scarp face was a high plasticity, saturated, dark brown clay. The undisturbed impervious material below the lower failure plane at elevation 435 ft NGVD was stable and was a stiff tan clay.

Some seepage between stations 77+50 to 78+00 existed before and after repairs were made along the downstream toe of the 1V:2.5H slope at approximate elevation 430 ft NGVD. Partial excavation of this area revealed that the path of the seepage is along the embankment fill/right abutment interface. Embankment material at this location was a stiff tan clay, plating over a moderately hard, tan sandstone foundation material. The seepage at this location does not appear to be affecting the stability of the embankment.

Station 81+30 to Station 82+85. This slide did not display any warning features such as cracking or bulging before failure and material displacement occurred. Guard posts and guard rails had to be removed before repairs began.

This slide, before repairs began, had a 12-ft exposed vertical scarp about 4 ft into the downstream crest, parallel to the centerline of the embankment. The scarp was arced and extended 65 ft down the slope from the crest edge. The scarp was 155 ft long at the crest and widened going down the slope to about 200 ft long at its lower failure plane at approximate elevation 435 ft NGVD on the 1V:2.5H slope. The slide debris rolled out and over the grass surface of the 1V:6H slope, stopping at elevation 414 ft NGVD just below the toe of the 1V:6H slope.

The slide debris and scarp face was composed of a high plasticity, very soft, saturated, dark brown (CH) clay. The undisturbed impervious material underlying the lower failure plane at elevation 435 ft NGVD was stable and was a moderately stiff tan clay.

Station 85+50 to Station 87+05. This slide, when first discovered on 21 December 1991, had a 5-ft exposed vertical scarp located midway between the crest edge and the toe of the 1V:2.5H slope. This slide did not display any cracking or bulging before failure and material displacement occurred.

During the excavation of the slide debris in August 1992, an arced longitudinal crack opened up on the centerline of the embankment crest between station 86+00 to station 87+00. As the slide debris excavation progressed below the first noted scarp and reduced the upper slope stability even further, a new crack would develop approximately 4 ft in elevation above the first scarp, followed by sloughing of that material. This cracking and sloughing occurred several times and continued up the slope until terminating at the crack on the centerline of the embankment crest. No cracking was found upstream of the embankment centerline.

The final exposed scarp on the crest of the embankment located on the centerline was 12 ft deep, extending 65 ft down the slope from the crest edge. The scarp was 150 ft long at the crest and widened going down the slope to about 250 ft long at its lower horizontal failure plane at elevation 436 ft NGVD on the 1V:2.5H slope. The slide debris rolled out and over the grass surface of the 1V:6H slope.

The slide debris and scarp face was a soft, very saturated, dark brown clay. The undisturbed material below the horizontal failure plane at approximate elevation 435 ft NGVD was a stiff tan clay.

Station 90+00 to Station 91+40. On 9 March 1992, an arced longitudinal crack approximately 140 ft long was discovered on the downstream edge of the embankment crest. The crack was arced and extended 15 ft down the slope from the crest edge. The crack was 140 ft long at the crest and widened going down the slope to 180 ft long at its lower limit on the 1V:2.5H slope. No further movement occurred until July 1992, at which time a 6-in. scarp was visible along the downstream crest edge. This slide did not display any bulging of slope material or any further vertical movement before repairs began in August 1992.

An inclined chimney and horizontal drainage blanket system exists in the embankment between station 87+00 to station 91+00. The top of the chimney is located at elevation 444 ft NGVD and the drainage outfall occurs at the downstream toe of the 1V:6H slope between elevation 418 ft NGVD down to elevation 415 ft NGVD. The slide movement and repairs did not disturb the drainage blanket.

The existing embankment material was removed for repairs between elevation 460 ft NGVD down to elevation 445 ft NGVD, 5 ft into the downstream crest. The material removed was a soft, saturated, dark brown clay. This entire section was replaced with a lime-stabilized fill. The toe of the drainage blanket at elevation 415 ft NGVD continued to be a wet area after repairs, indicating that the blanket is still functioning.

Station 93+00 to Station 94+00. On 9 March 1992, an arced longitudinal crack approximately 100 ft long located 4 ft into the downstream crest was discovered. The crack extended 5 ft down the slope from the crest edge. No further movement occurred until June and July 1992, at which time bulging on the 1V:2.5H slope and vertical movement at the crest along the crack was observed.

This slide, before repairs began, had a 10-ft exposed vertical scarp located 4 ft into the downstream crest parallel to the embankment centerline. The scarp was 100 ft long on the crest, widening and extending 55 ft down the slope to 200 ft long at its lower failure plane at approximate elevation 435 ft NGVD on the 1V:2.5H slope.

The slide debris rolled out and over the grass surface of the 1V:6H slope, stopping at elevation 420 ft NGVD. The slide debris and the material of the exposed slickensided face of the scarp was soft, saturated, high plasticity, dark brown clay. The undisturbed impervious material below the lower horizontal failure plane at approximate elevation 435 ft NGVD was stable and is a stiff tan clay.

Station 95+50 to Station 97+50. On 9 March 1992, project personnel discovered a 200 ft long section of longitudinal alligator-type cracking, located 5 ft into the downstream crest parallel to the embankment centerline. No further movement occurred until June and July 1992.

During the months of June and July 1992, this area developed a 2-ft scarp on the crest which extended down the slope from the downstream crest edge. Bulging on the 1V:2.5H slope continually increased in size as the scarp height increased. The exposed scarp was 10 ft deep when no further movement was noted before repairs began. The scarp was 200 ft long on the crest and widened to 250 ft long at its lower failure plane at elevation 435 ft NGVD on the 1V:2.5H slope.

The slide debris and exposed scarp face material indicated that this area had undergone previous repairs. The scarp face consisted of dark brown clay, decomposed vegetation, base course material, sand, empty lime sacks,

unmixed lime, tan clay, and some crushed limestone up to 8 in. in diameter. The undisturbed impervious material below the lower horizontal failure plane at elevation 435 ft NGVD was stable and is a stiff tan clay.

Station 101+00 to Station 104+00. The first section of this slide, from station 101+00 to station 102+35, did not display any warning features such as cracking or bulging before failure and displacement of the material occurred on 21 December 1991.

A longitudinal crack, located 1 ft into the downstream crest from station 102+35 to station 104+00, was also observed on December 1992. This crack, which was an extension of the slide at station 101+00 to station 102+35, did not display any further movement until 26 January 1992.

On 26 January 1992, from station 101+00 to station 104+00 displayed a 2 ft deep vertical scarp located 1 ft into the downstream crest. The slide debris rolled out and over the grass surface of the 1V:2.5H slope and the 1V:6H slope, stopping at elevation 428 ft NGVD.

During the months of June and July 1992, the scarp continually increased in depth to a maximum depth of 20 ft, 5 ft into the downstream crest. The slide debris stopped at elevation 414 ft NGVD.

Before repairs began, the scarp was 300 ft long on the crest, widening and extending 75 ft down the slope to 400 ft long at its lower failure limit at elevation 430 on the 1V:2.5H slope. The slide debris and scarp face indicated that this area had undergone previous repairs. The scarp face consisted of dark brown clay, decomposed vegetation, base course material, sand, empty lime sacks, unmixed lime, tan clay, and some crushed limestone up to 8 in. in diameter. The undisturbed impervious material below the lower failure plane at elevation 430 ft NGVD was stable and is a stiff tan clay.

Testing

Following the slides which occurred in January and June 1986, it was decided to sample the slide and potential borrow areas for laboratory testing to determine optimum lime content. Three samples from selected slide areas and two samples from potential borrow areas were taken to the laboratory and subjected to lime modification to determine optimum lime modification content. The samples were mixed with 0 to 10 percent hydrated lime and then subjected to a pH test. The optimum lime content for all the samples was found to be in the range of 4 percent. In order to account for uncertainties with the mixing process during construction, the amount of hydrated lime for stabilization was specified as 6 percent.

Repair Methods

Historically, slide areas on the Bardwell Lake Dam have been repaired by the same method since 1973. This method consisted of removing the slide debris material, mixing lime into the material, and recompacting the lime-soil mixture back to the original 1V:2.5H slope template. Previous crack and slide repair efforts have only been marginally successful, as some slides have reoccurred in places previously repaired.

The slope stability could be improved by constructing a new flattened slope over the existing 1V:6H slope. It was decided before repair of the 1991/1992 slides to flatten the existing 1V:2.5H slope to a 1V:3.6H slope. The toe of the new improved slope is at elevation 416.0 ft NGVD from station 78+00 to station 100+00. Approximately 65,000 cu yd of borrow material was required to construct the new improved slope. Some of the slide debris and existing embankment material was found to be well above optimum moisture content during excavation and unstable. This material was excavated, processed to acceptable moisture levels, and then recompacted in the new slope below elevation 450 ft NGVD.

Due to the history of longitudinal cracking on the downstream half of the embankment, this section of the embankment from station 78+00 to station 100+00, elevation 450 ft NGVD to 460 ft NGVD, was removed and replaced with lime-stabilized fill. The repair area between station 100+00 and station 105+00, which also is the new slope transition into the existing slope, was lime stabilized in its entirety. A total of 1,262 dry weight tons of lime was used to produce approximately 15,000 cu yd of soil-lime mixture at 6.5 percent lime by dry weight of soil.

The downstream half (10 ft) of the embankment service road required 1,131 tons of new flexible base course material to restore the road to finish elevation 46 ft NGVD between station 78+00 and station 105+00. The downstream portion of the service road between station 105+00 and station 158+00 was also watered, reshaped, and recompacted.

1992 Slide repair costs. (all work performed by equipment rental contract)

a. Remove/replace electric lines and power poles		$ 5,037.41
b. Remove/replace guard post and railing		25.00
c. Embankment slide repair		569,797.50
d. Plow, fertilize, and plant wheat		10,565.00
	TOTAL	$585,424.91

1992 Slide repair quantities.

a. Borrow for new slope 65,000 CY

b. Lime-stabilized fill 15,000 CY

c. Strip/replace topsoil 28,000 CY

d. Lime 1,262 Tons

e. Base course material 1,131 Tons

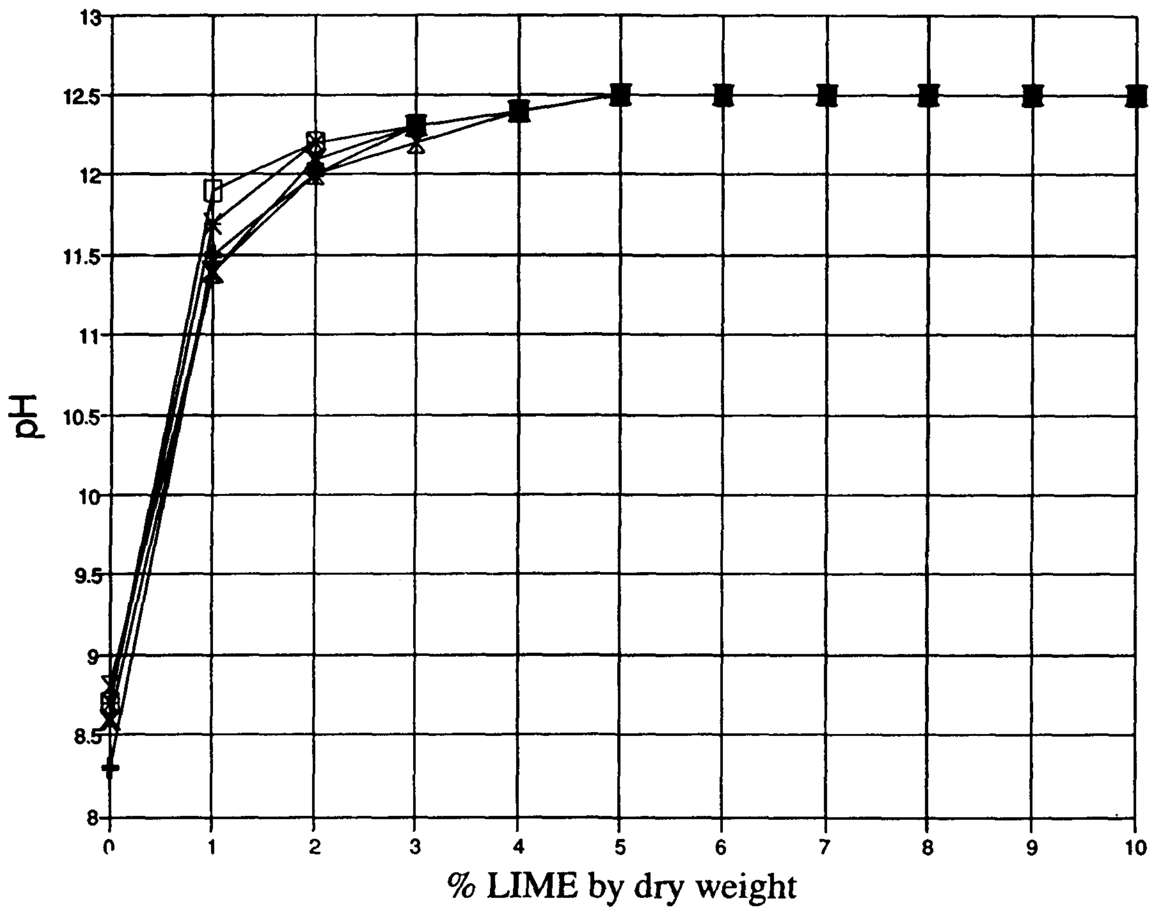
Slope stability. The occurrence of skin slides indicates the existing 1V:2.5H slope has a factor of safety (with respect to shallow skin slides) of less than 1, under certain conditions. Such conditions mentioned previously are long dry spells followed by wet rainy periods when the material becomes saturated and desiccation cracks on the surface of the embankment crest become filled with water. Testing of embankment record samples shortly after construction indicates the materials exhibit a drained shear strength of approximately 16 deg. Analyses of the slope using Spencer's method indicate the slope to have a factor of safety as low as 0.8, assuming a water-filled desiccation crack of 5 ft in depth. The slides typically occur after long dry periods followed by a wet rainy period. The crack fills with water before the soil mass as a whole can swell up and close the crack. To improve the slope stability, the slope must be flattened and/or the strength of the soil must be increased. The new 1V:3.6H slope increases the factor of safety to 1.2, assuming the soil shear strength is constant. However, the newly constructed notched-in, lime-stabilized crest section will have a higher strength than the existing material. Additionally, the lime stabilized notch should experience lower volume changes, thus lowering the possibility of deep desiccation cracks which could fill with water.

Future work. The remaining 1V:2.5H downstream portion of the upper slope is scheduled to be flattened to 1V:3.6H in FY94. Project personnel will continue to monitor the embankment crest for cracking and potential slides.

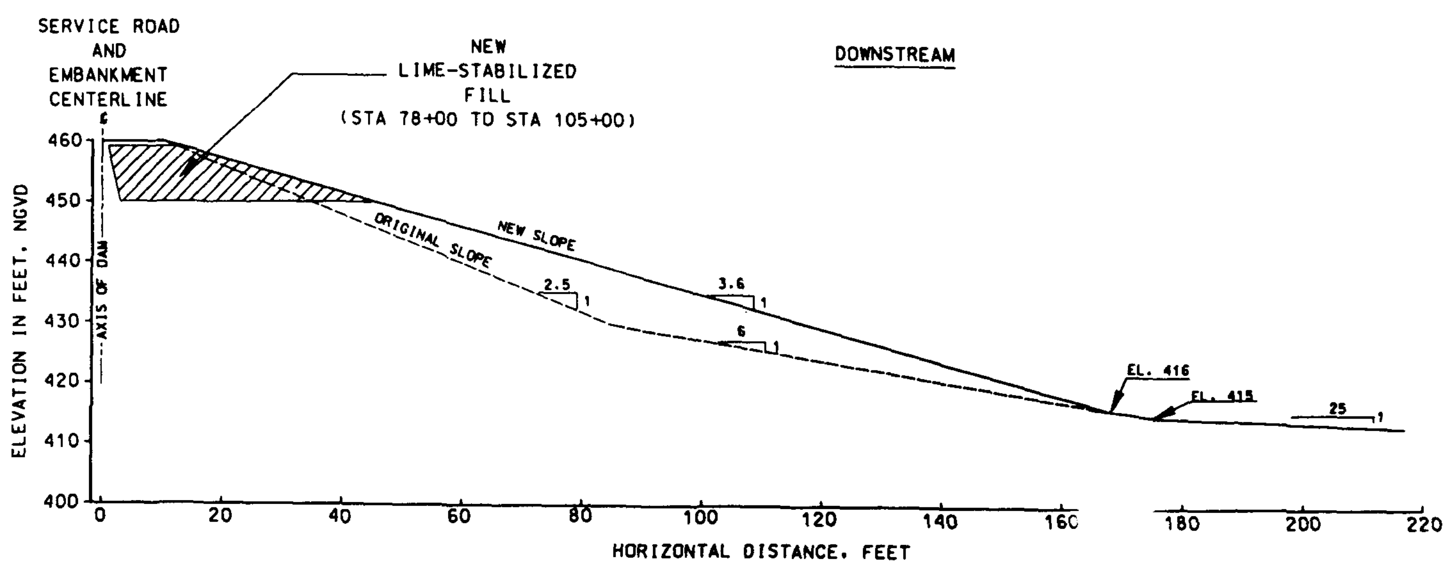
Table 1
Slide History - Bardwell Lake

Date	Location	Slide Limits Upper / Lower	Scarp Height	Crest Encroachment
10 Sep 68	Sta 78 to Sta 118	Crest Cracking	N/A	N/A
25 Apr 73	Sta 95+50 to Sta 100+50	EL 450 to EL 435	4-ft Scarp	None
25 Apr 73	Sta 101+75 to Sta 104+25	EL 453 to EL 430	6-ft Scarp	None
25 Apr 73	Sta 114+50 to Sta 115+50	EL 450 to EL 435	4-ft Scarp	None
3 Feb 75	Sta 89+40 to Sta 90+25	EL 457 to EL 440	4-ft Scarp	None
3 Feb 75	Sta 98+75 to Sta 100+50	EL 460 to EL 430	10-ft Scarp	2 ft into crest
23 Jan 75	Sta 105+25 to Sta 106+05	EL 450 to EL 435	3-ft Scarp	None
23 Jan 80	Sta 83+00 to Sta 85+00	EL 460 to EL 440	2-ft Scarp	2 ft into crest
23 Jan 80	Sta 97+50 to Sta 98+50	No slide observed	0.5-ft Scarp	N/A
23 Jan 80	Sta 82 to Sta 118	Crest Cracking	N/A	N/A
17 Jun 81	Sta 85+00 to Sta 87+00	EL 460 to EL 440	2-ft Scarp	1 ft into crest
17 Jun 81	Sta 101+00 to Sta 103+00	EL 457 to EL 440	1-ft Scarp	None
17 Jun 81	Sta 109+50 to Sta 110+75	EL 460 to EL 440	10-ft Scarp	2 ft into crest
17 Jun 81	Sta 94+50 to Sta 95+50	EL 460 to EL 440	Cracking	None
17 Jun 81	Sta 95 to Sta 135	Crest Cracking	N/A	N/A
17 Jun 81	Sta 90+50 to Sta 91+50	Riprap depression	N/A	N/A
17 Jun 81	Sta 96+50 to Sta 97+50	Riprap depression	N/A	N/A
3 Jan 86	Sta 97+50 to Sta 99+36	EL 460 to EL 440	10-ft Scarp	1 ft into crest
05 Jun 86	Sta 109+50 to Sta 111+00	EL 448 to EL 430	10-ft Scarp	None
26 Feb 87	Sta 106+50 to Sta 108+50	EL 460 to EL 430	4-ft Scarp	1 ft into crest
15 Oct 87	Sta 95+00 to Sta 97+40	No slide observed	0.5-ft Scarp	None
15 Oct 87	Sta 103+00 to Sta 105+00	No slide observed	0.5-ft Scarp	None
20 Dec 91	Sta 79+00 to Sta 80+00	EL 460 to EL 430	12-ft Scarp	8 ft into crest
20 Dec 91	Sta 81+30 to Sta 82+85	EL 460 to EL 430	12-ft Scarp	4 ft into crest
20 Dec 91	Sta 85+50 to Sta 87+05	EL 450 to EL 430	12-ft Scarp	10 ft into crest
20 Dec 91	Sta 101+00 to Sta 102+35	EL 460 to EL 430	20-ft Scarp	6 ft into crest
26 Jan 92	Sta 102+35 to Sta 104+00	EL 460 to EL 430	20-ft Scarp	6 ft into crest
9 Mar 92	Sta 90+00 to Sta 91+40	No slide observed	0.5-ft Scarp	1 ft into crest
9 Mar 92	Sta 93+00 to Sta 94+00	EL 460 to EL 435	10-ft Scarp	[illegible] ft into crest
9 Mar 92	Sta 95+50 to Sta 97+50	EL 460 to EL 435	10-ft Scarp	5 ft into crest

BARDWELL DAM
EMBANKMENT SLIDE REPAIR



Graph represents laboratory testing of samples to determine optimum lime content.



TYPICAL SECTION
STATION 78+00 TO STATION 100+00
BARDWELL DAM
AUGUST 1992

Double Lime Application for Levee Stabilization[1]

Mark S. Alvey
St. Louis District

[1] Written paper not provided (see next page for alternate paper).

Levee Slide Repairs Using a Double Application of Hydrated Lime[1]

Mark S. Alvey
St. Louis District

Levees constructed of highly plastic soils are very susceptible to failure due to the continuous cycles of swelling and shrinking which leads to desiccation and cracking. Repair methods used in the past which have failed include:

a. Simply pushing the failed material back into place.

b. Backfilling the slide depression with new materials.

c. Discing hydrated lime into the surface materials and reshaping the levee.

d. Constructing a drainage layer in the levee to provide internal drainage.

Most levees in the St. Louis District were constructed in the 1950's using clay materials. Typically, the levees have 1 vertical on 3 horizontal side slopes with a 20-ft wide crown and range in height between 20 and 25 ft. There are areas where highly plastic clays were used with the typical cross section and the levees have continually failed. Using the infinite slope analysis for the long-term drained condition with clays having effective shear strengths of cohesion = 0 psf and $\phi' \leq 20°$, the factors of safety are at or below one. These soils should have been constructed with slopes no steeper than 1 vertical on 4 horizontal. Unfortunately, additional right of way is very difficult to obtain and the levee districts are unable to pay for their share of reconstruction costs under the present 25 percent local cost share formula.

Modification of the soils was determined to be the best alternative to repair the levee slides. Although, it does not address the total levee structure which is evident by an average of 12 new slides occurring each year. Soil

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[1] *Proceedings of the SAD-ORD Geotechnical Conference '93*, Orlando, FL, U.S. Army Engineer District, Jacksonville.

modification with hydrated lime was selected after some study which found fly ash and Code L (lime manufacturing by-product) had limited results. A very good reference in these types of repairs is a Waterways Experiment Station Technical Report GL-79-2, "Use of Lime in Levee Restoration," by Frank C. Townsend dated September 1979. The St. Louis District has found it is beneficial to specify double applications of hydrated lime when:

a. Soil plasticity index is greater than 40.

b. Soils are difficult to break down into workable sizes.

c. Single application exceeds 4 percent hydrated lime and mixing difficulties increase.

d. Uniform blend of hydrated lime is desired.

The construction procedure and sequence for double application hydrate lime are specified in the construction repair contracts. The basic procedures and sequences are presented below. Remove the top 12 in. of topsoil which remains untreated and is replaced upon completion of the repair. An inspection trench is excavated into the slide to establish the failure surface location. Excavate the failed material plus 2 ft behind the identified surface. Place the excavated soil in 10-in. lifts within a stockpile area where half of the required lime is applied and blended into the soil followed by successive 10-in. lifts of treated material. Treat the exposed bottom of the repair with half of the required lime before backfilling begins. The second application of hydrated lime is blended into the stockpiled treated material on each successive lift. Apply the remaining half of the required lime uniformly and blend into the treated soil within the stockpile to a depth of 10 in. This 10-in. layer is then excavated, transferred into the repair area and compacted within 8 hr of applying the lime. Upon completion of the repair area, the topsoil is replaced, fertilized, seeded, and mulched.

Blending hydrated lime into wet and highly plastic clays is a difficult task. Self-propelled high speed soil pulverizer equipment with a minimum horsepower of 318 is recommended. The equipment is capable of blending materials down to 12 in. and only requires one or two passes. Deep cutting disc equipment will work on soils with moderate plasticity; although, thin lifts and numerous passes are recommended to thoroughly blend the hydrated lime with the soils.

Use of Geogrids for Levee Slope Stability Problems[1]

Dennis W. Abernathy
Memphis District

[1] Written paper not provided.

Lime Stabilization and Rock-Fill Trenches[1]

George L. Sills
Vicksburg District

[1] Written paper not provided (see the following alternate papers).

Lime Stabilization of Levee Slopes[1]

Robert L. Fleming, Jr., George L. Sills and Edwin S. Stewart, Jr.
Vicksburg District

Abstract

The U.S. Army Engineer District, Vicksburg, has developed a procedure for using lime in repairing slough slides that occur in the riverside slope of the Main Line Mississippi River Levees. Since 1964, approximately 450 main line levee slides have been repaired. After studying the cause and occurrence of these slides between 1968 and 1982, a procedure was developed for using lime to repair the slides. The history and results of those slide studies, the description of the repair procedures, and the performance of repaired slides to date are discussed in this paper.

Introduction

The U.S. Army Engineer District, Vicksburg, has approximately 1,670 miles of levee to maintain. Of this, 460 miles are a part of the main line Mississippi River Levee system located in the states of Arkansas, Louisiana, and Mississippi. The levees in the main line system range in height from 25 to 40 ft. In addition to the main line system, there are approximately 1,210 miles of backwater and headwater levees that range in height from 5 to 25 ft. All of the main line levees and a large percentage of the backwater and headwater levees were constructed with riverside slopes that consist of clays (CH) and (CL) to limit through seepage. Shallow slough slides have been occurring along the riverside slopes of these main line levees for the last 60 years. Between 1964 and 1982, approximately 200 to 225 slides were repaired. In 1979 alone, a total of 41 slides were repaired and at that time represented the largest number of slides to occur in a single year.

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[1] *Proceedings of the Second Interagency Symposium on Stabilization of Soils and Other Materials*, 2-5 November 1992, Metairie, Louisiana, pp 5-15 to 5-22.

The typical slough slide usually develops in the riverside slope which is normally steeper than the landside slope. Most riverside slopes are a 1V on 4H slope. The typical slide can be defined as a shallow slide whose maximum depth to the slip plane varies between 4 and 8 ft and whose failure is triggered by heavy rainfall after an extended period of weathering. Weathering results from desiccation and causes strains to be induced during the seasonal shrinking and swelling process. The zone of weathering that develops usually extends to a depth of 5 to 7 ft. The process and its effect on slide development is described in detail later in this paper. The slides occur primarily between the riverside crown and a point midway down the slope and range in length from 100 to 300 ft along the levee. A typical slide is shown in section and plan in Figure 1.

This paper describes the efforts to identify the causes of these slides and the methods, both past and present, that are used to repair them.

Background

Prior to the early 1980's, the method of repairing these slides normally consisted of excavating the failed material and rebuilding the slopes on a flatter angle with new material from a borrow source. In some cases a berm was used to repair the slides. Although not required by the 1947 Mississippi River Commission Levee Code, it had become the policy of the Vicksburg District to construct a standard 40-ft-wide riverside buttress berm on all levees 25 ft or greater in height. This policy was unofficially adopted in 1962 in an effort to prevent slough slides. In the late 1960's and 1970's, the slides seemed to become more numerous. Also, obtaining rights-of-way and additional sources of borrow was becoming more and more costly and time consuming. It was clear that a more cost-effective method of repairing the slides was needed.

The first organized effort to evaluate slough slide failures began in 1968. Several slides were trenched and a limited laboratory testing program was initiated in an attempt to determine soil parameters or site conditions unique to the areas experiencing slough slide failures. The results of these observations were summarized in an unpublished report by Larry Cooley, who at the time was Chief of the Foundation and Materials Branch. Following this report, the Vicksburg District trenched several more of these slides to obtain "undisturbed" soil samples and to observe fracture planes, crack distribution, and material composition. Observed similarities between materials and macrostructure in these slides and those described in the literature relating to long-term failures in cuts in stiff, fissured, highly plastic clays seemed to indicate that a time-dependent weakening of these levee materials was occurring due to seasonal shrinking and swelling. Skempton (1978) suggested that failure would occur just before reaching the value of fully softened strength, which is equal to the peak strength of the remolded normally consolidated clay.

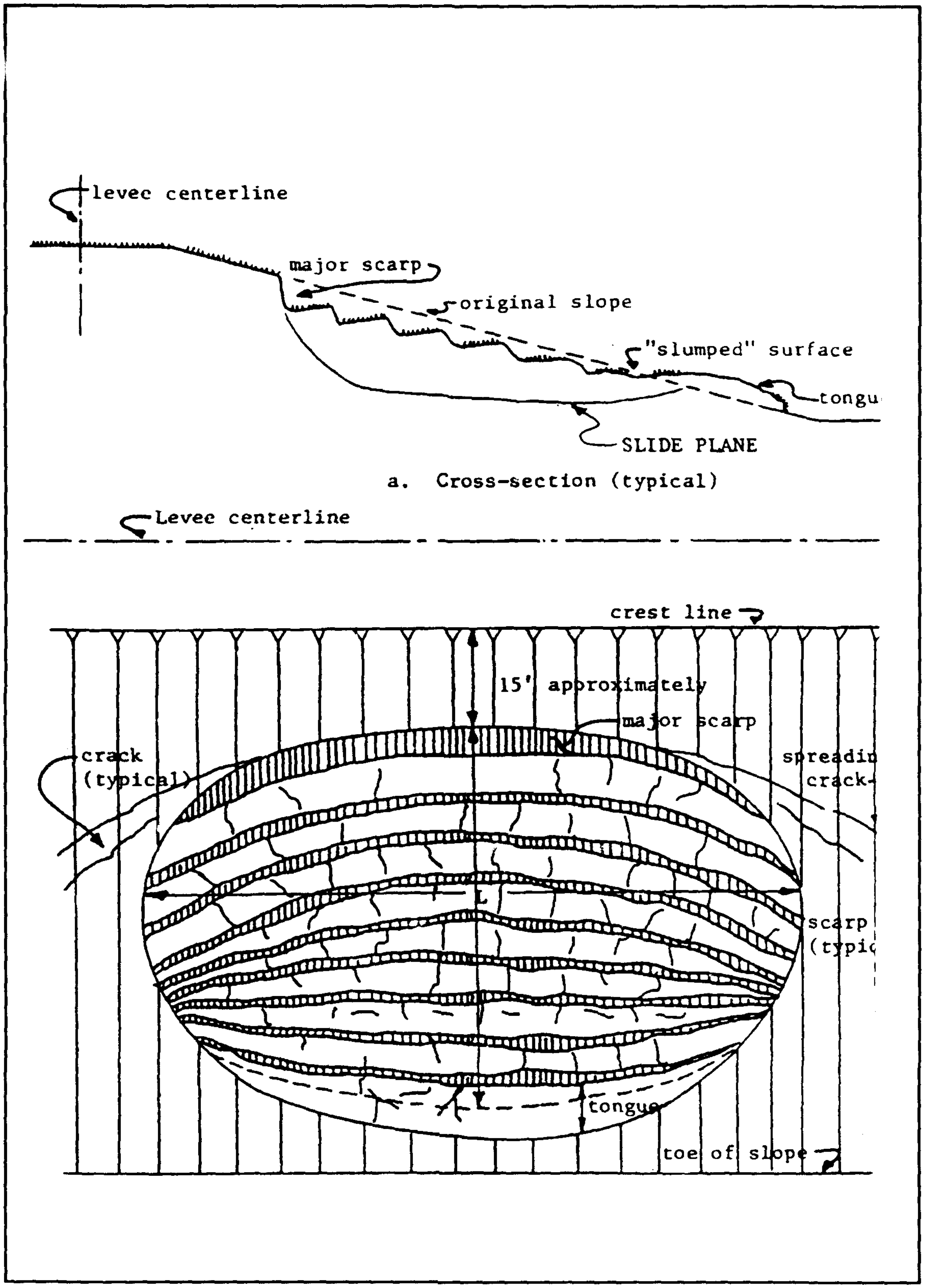

Figure 1. Typical slough slide: (a) cross section; (b) plan view

George Sills published a Master's Thesis (1981) entitled, "Study of Long-Term Failure in Mississippi River Levee Material." This helped to add emphasis to find a solution. Using Cooley's unpublished observations and Sills' thesis as a starting point, the Vicksburg District embarked upon an extensive field and laboratory study to relate slide susceptible areas to some soil property that designers could use in levee enlargement design to minimize the occurrence of these slides and to also use in determining a repair for these slides that would surely occur on the 460 miles of existing main line levees.

Mechanisms of Slough Slide Development

It was evident from data collected that the slough slides occurring along the levee systems were a result of long-term reduction in shear strength. Very little documentation existed concerning long-term failures in compacted highly plastic clays. However, similarities between failures in compacted highly plastic clays and cut slopes in stiff fissured clays suggest meaningful comparisons of the mechanism of failure can be made.

The reduction in strength in the levee embankment apparently results from weathering effects and strains induced by seasonal shrinking and swelling. During dry periods, shrinkage cracks open to a depth of 5 to 7 ft. These cracks expose the interior of the mass allowing deeper desiccation to occur and fissures to form due to irregular shrinking. Subsequently, water percolates through these cracks and fissures causing the material to swell and slake. Laboratory tests have shown that this slaking results in a permanent increase in volume which must be accompanied by increase in stress. These stress increases are concentrated along discontinuities and local over stressing occurs forming segmented slickensides in zones experiencing the largest strains. The discontinuous slip surfaces begin to interconnect at the toe of the slide and advance into the slope as the slide develops. It has been noted that the maximum depth of the slide coincides with the depth of desiccation in most cases.

The slough slides appear to be triggered by heavy rainfall after an extended period of drying. The extensive network of cracks and fissures developed by years of weathering increases the mass permeability of the embankment. When these cracks fill with water, the exposed surfaces along the cracks and fissures soften, reducing the shear strength along these discontinuities. Piezometric data obtained from this study indicate that a perched water table forms above the intact clay zone located below the weathered zone. The increase in driving weight and accompanying softening of the exposed clay combined with the progressive loss of shear strength due to long-term seasonal shrinking-swelling effects result in a slough failure.

Additional Field Investigation

Sills (1981) recommended the Vicksburg District obtain additional riverside blanket borings with the intent to delineate reaches most susceptible to slough slides. It was felt that by determining the Atterberg limits of the levee material a delineation could be made since these indices are indications of the shrink swell characteristics of a soil. In an effort to identify problem areas and to search for more potentially cost-effective methods of repair, an extensive riverside blanket boring program was conducted along the east bank Mississippi River levees in 1982. Over 919 borings were made to assess the extent of highly plastic clays existing on the riverside slopes. Samples were recovered for classification and Atterberg limit determinations. Borings were taken at a maximum spacing of 1,000 ft. The reach investigated began just north of Vicksburg, Mississippi, and extended up river to the limit of the Vicksburg District, which is just south of Memphis, Tennessee. The spacing was reduced to 500 ft in reaches where bulging or known sliding had occurred on the riverside slope. Borings were made with a spiral auger to a minimum depth of 5 ft. Stratum changes were noted and general samples were collected at depths of 3 and 5 ft.

It was found that approximately 50 percent of the area studied consisted of highly plastic clay (CH). From these data it appeared that a limiting value of PI may be associated with materials susceptible to slough slides on these 1V on 4H slopes. These data indicated that no slides have occurred in areas where the PI is less than 27 and very few where the PI is between 27 and 40. It should be further noted that these PI values do not necessarily correspond to the material involved in the slough slides since the data were obtained after many of these slides had been repaired and, in some cases, were extrapolated from borings adjacent to the slide area. Figure 2 is a plot of Atterberg limit indices of materials recovered directly from slides which indicates that materials susceptible to slough slides may be characterized as having a liquid limit greater than 60 and a PI greater than 40. Sills et al. (1983) reported these findings in a technical report prepared by the Vicksburg District dated 1983.

Additional Studies

Additional research into this slide problem has revealed that in areas protected from the weathering process, no slide will occur. The best example of this is on a rock protected dam slope where the bedding gravel and rock tend to protect the soil from weathering on the lake side. Very few slides occurred on the lake side where as numerous slides occurred on the flatter land side. The protection riprap and bedding gravel offer from the weathering process was clearly demonstrated in the Lake Chicot Pumping Plant outlet channel near Lake Village, Arkansas. The Lake Chicot Pumping Plant was built in the Main Line Mississippi River Levee across the river from Greenville, Mississippi. For underseepage control, soil bentonite slurry trench cutoff was constructed. The structure is tied to the slurry trench cutoff with a neoprene

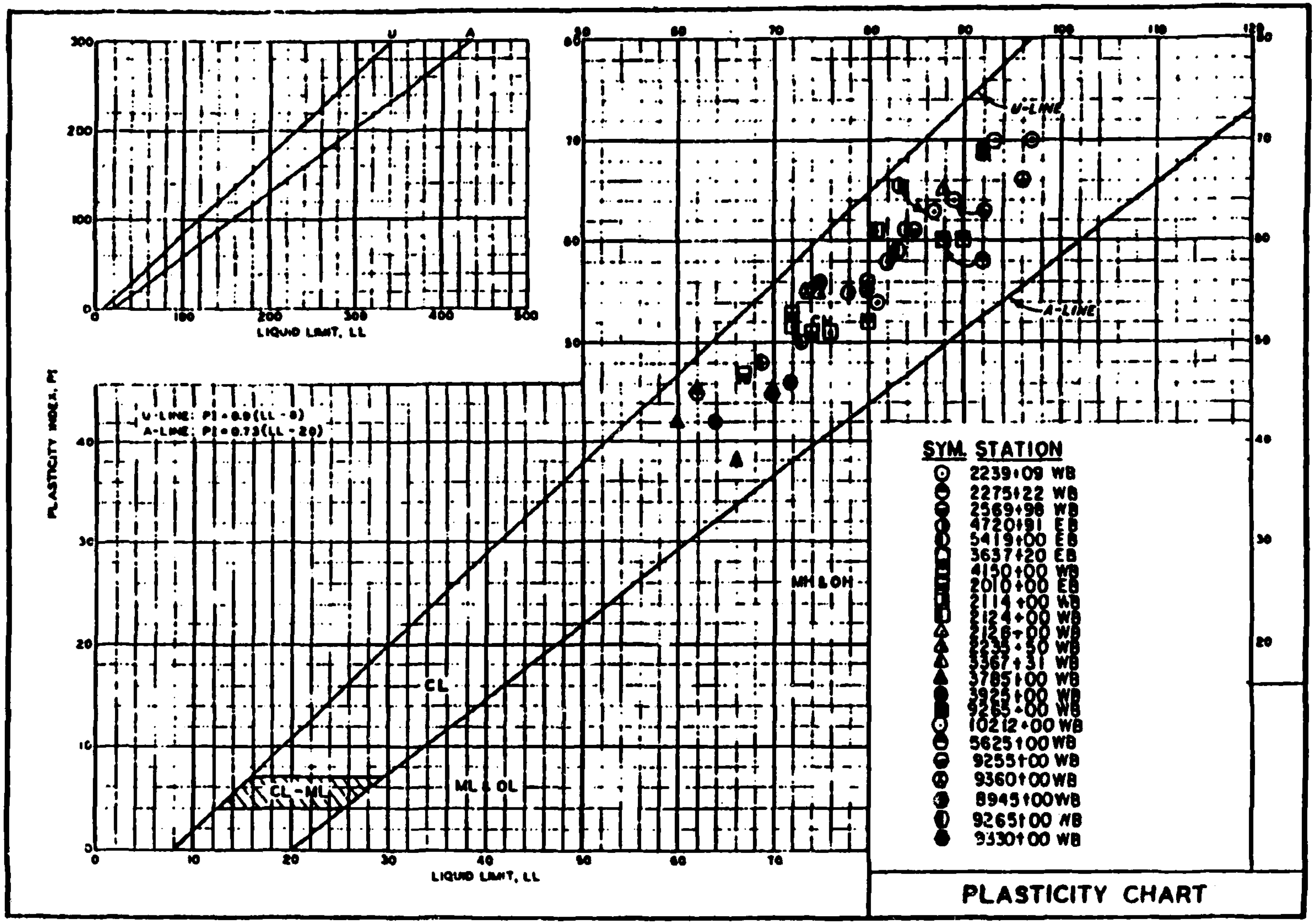

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Figure 2. Limit data from slough slides

coated nylon fabric that is connected to the structure and anchored into the top of the slurry trench.

The impervious fabric is covered with a 5-ft-thick clay (CH) blanket. In the riverside outlet channel, the fabric and clay blanket were placed on a 1V on 3H slope. The channel slopes nearest the structure were covered with bedding gravel and riprap. There was a portion of the channel slope capped by the impervious fabric and clay blanket that was not covered with bedding gravel and riprap. In 1986, less than one year after completion of the project, a slide occurred in that portion of the clay blanket that was not covered with gravel and riprap. The slip plane was at the interface of the fabric and clay material. The slide was repaired by replacing the failed material with similar type material. Bedding gravel and riprap were then placed on the remainder of the channel that had the clay blanket covering the impervious fabric. Since that time, we have experienced no further problems. This has been attributed to the belief that the bedding gravel and riprap greatly reduced the weathering process by maintaining a fairly constant moisture content in the clay blanket.

Slide Repair Using Lime

Since 1985 the Vicksburg District has been using lime where possible to repair slough slides because material having a PI of less than 40 is usually not available within a reasonable haul distance of slide repairs. So far, lime has been used in the repair of approximately 142 slides of various sizes.

The first step in the repair process is to obtain representative bag samples from each slide. A suitable area is chosen in the slide itself and all topsoil is removed. A sample of the underlying soil is placed in a bag and transported to the laboratory for soil tests which will be used to determine the amount of lime to be used in the repair.

In the laboratory, Atterberg limits are determined for the soil in its natural state and with varying amounts of lime added. The results of these tests are plotted and the optimum amount of lime is chosen. The optimum percentage of lime is the minimum amount that will lower the plasticity index below 40 and produce a pH of approximately 12. Once the required lime percentage is determined the remaining portion of the bag sample is prepared for a compaction test to determine the optimum density and water content with the lime added. The results of the compaction tests are plotted and used for quality control in the field.

Construction Procedures

The first order of work in the field is to remove all topsoil from the slide. The topsoil is stockpiled near the slide and will be replaced when the repair has been completed.

The slide material is then excavated to a point "about 2 ft" below the failure plane. The slip plane is usually very easy to find in the field. When the size of the slide being repaired permits, the lime-soil mixture can be processed in lifts in the embankment and compacted; therefore, the excavated material can be stockpiled at the toe of the slide. If the slide being repaired is small, a soil processing area located adjacent to the slide may be required. The topsoil in the slide area should be removed from the soil processing area and stockpiled for reuse. Excavated material from the slide should be spread evenly over the soil processing area in a lift of 6 to 8 in. in thickness.

The specified amount of lime should be spread evenly over the surface of the soil to be mixed. After application of the lime, a light application of water should be used to prevent dusting and achieve a good distribution of the lime when mixed. Several passes of a rotary pulverizer is then used to mix the soil and lime. In clays with a very high plasticity index, a double application of lime must be used to thoroughly incorporate the lime with the soil. If a double application of lime is used, the first application should be one-half the total amount of lime to be applied. It should be mixed by at least one pass of a rotary pulverizer and then sealed with a steel roller and allowed to cure for 48 hr. After curing, the remainder of the lime should be added and mixed.

After final mixing of the lime with the soil, the mixture is placed in the embankment in 6- to 8-in. lifts and compacted by at least three passes of a dozer. After the required number of passes, the water content and density are checked. If adequate density is not obtained, the water content is adjusted or additional compaction is applied. When placement of the lime-soil mixture is completed, the surface is dressed to final grade.

Topsoil is then replaced and fertilizer is uniformly distributed over the surface and worked into the soil by light disking. After fertilizing, grass seed is spread over all disturbed areas.

The surface is then sealed with a steel roller to retain moisture for better seed germination. The use of lime in the repair of slough slides has thus far proved to be much more economical and effective than flattening the slopes with berms. The slides are reconstructed to the original grade using material from the slide itself thereby reducing the right-of-way requirements and the need for additional borrow.

Based on the experiences described above that indicate riprap and bedding gravel placed on a clay slope tend to greatly reduce the weathering process, the Vicksburg District in 1990 began to use a modified procedure using lime to repair these slides. To date, a total of 14 slides have been repaired using a capping process. The slide area and failure plane are removed in the same manner as described above. The major difference is that only the outer 3 ft of soil on the reconstructed slope are lime treated instead of treating the entire mass. We are presently treating approximately 10 percent of the slides repaired each year in this manner to test the effectiveness of this capping

process. It is our opinion that this will provide a protective cap of treated soil that will be resistant to the weathering process that cause the slides to occur.

Conclusions

The use of lime has proved to be a cost-effective method of repairing slough slides on the Main Line Mississippi River Levees. Costs associated with obtaining rights-of-way and extra borrow material has been eliminated since all work can be carried on in the existing rights-of-way limits. The average cost of a slide repair using lime has been reduced to approximately $20,000 to $25,000. If the process of capping proves to be successful, these costs can be reduced even further.

Acknowledgements

The authors would like to express their thanks to the U.S. Army Corps of Engineers, Vicksburg District, for supporting these efforts. The authors would also like to thank Mr. Larry Cooley and Mr. Alexis E. Templeton for their efforts as a part of this study, Mr. Mckinley Harris for preparing the figures, and Mrs. Sue L. Richardson for typing this paper.

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Slope Stabilization with Stone-Fill Trenches[1]

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Vicksburg District

Abstract

The U.S. Army Engineer District, Vicksburg developed a different method of slide stabilization using gravel-fill or stone-fill trenches. A number of slides have been successfully stabilized with this method. The design, construction, and performance monitoring of one of these repairs is discussed, along with a history of the slide, description of the soils, and mechanism of failure. The applications of this method of slide stabilization are also evaluated.

Introduction

The U.S. Army Engineer District, Vicksburg, has successfully used gravel-fill or stone-fill trenches to stabilize a number of slides. In 1982, a long shallow translatory slide in a medium to stiff highly plastic clay was stabilized with a technique using trenches excavated below the slip plane and partially filled with a washed gravel aggregate (Wardlaw 1984). The slide which had closed one lane of a county road and had destroyed a timber bridge, has remained essentially stable since it was stabilized in 1982. Only a small amount of strain cracking in the pavement has been evident. Since that time a number of other slides have been successfully stabilized using washed gravel and more recently quarry run graded stone. Two of the more recent applications involving the use of stone-fill trenches to stabilize slides were on the inlet channel at John H. Overton Lock and Dam on the Red River Waterway and on the Ouachita River at Rilla, Louisiana. The Rilla slide was located 9 miles downstream of Monroe, Louisiana, at approximate River Mile 166 on

[1] *Stability and Performance of Slopes and Embankments-II*, 29 June - 1 July 1992, Berkeley, California, American Society of Civil Engineers, Geotechnical Special Publication No. 31, New York, Vol. 2, pp 1382-1394.

the Ouachita River and is shown in plan on Figure 1. The slide had involved a portion of the Ouachita River Levee and was considered to be an endangerment to the flood control system. The repairs were made under emergency conditions in December 1988. A method of repair, not materially affected by the weather conditions, that could be completed prior to the next high water season was selected. In addition, the repair method could not extend into the navigation portion of the channel.

This paper summarizes the nature and history of the Rilla Slide, the design of the stabilization measures, construction of the stone-filled trenches and the results of postconstruction monitoring.

Background

The slide location, in the outside channel slope of a meander of the Ouachita River, has been subjected to active scour. The lower portion of the river channel slope consists of highly erodible sands and silty sands. The upper portion of the channel slope consists of clays and silts. Prior to 1983, the levee had been set back because of bank caving problems in this reach. In the summer of 1983 scouring had caused an instability in the upper clay slope. During low river stages, the slide scarp had propagated into the riverside toe of the previous levee setback. At this time, further setback of the levee was not considered feasible because of existing houses and structures located landside of the levee toe. A field and laboratory investigation program was initiated to design a section to stabilize the slide and to stop further degradation of the river bank by scour. The recommended repair consisted of a large stone toe dike and a sand berm. The stone dike would serve to buttress the sand fill as well as provide scour protection. Construction of the dike and sand fill was initiated in the fall of 1984. Because of problems, such as extended high water and an extremely compressed work area, the slide was never properly stabilized and continued to slowly move. By the summer of 1988, the slide movement had reached a point where it was considered an endangerment to the levee and needed to be repaired prior to the next high water in the spring.

Because of these problems only those methods that would not be affected by fluctuation of the river, by wet weather conditions, or impact the navigation channel were considered. This led to the use of stone-fill trenches to stabilize the slide. Because of the emergency nature of the project and problems with the previous construction, the specifications were very specific in terms of the order and the compressed timing of the work. Construction was initiated on 7 December 1988 and was completed on 19 December 1988. As of the time of this paper in December 1991, there has been no evidence of further movements along the slip plane.

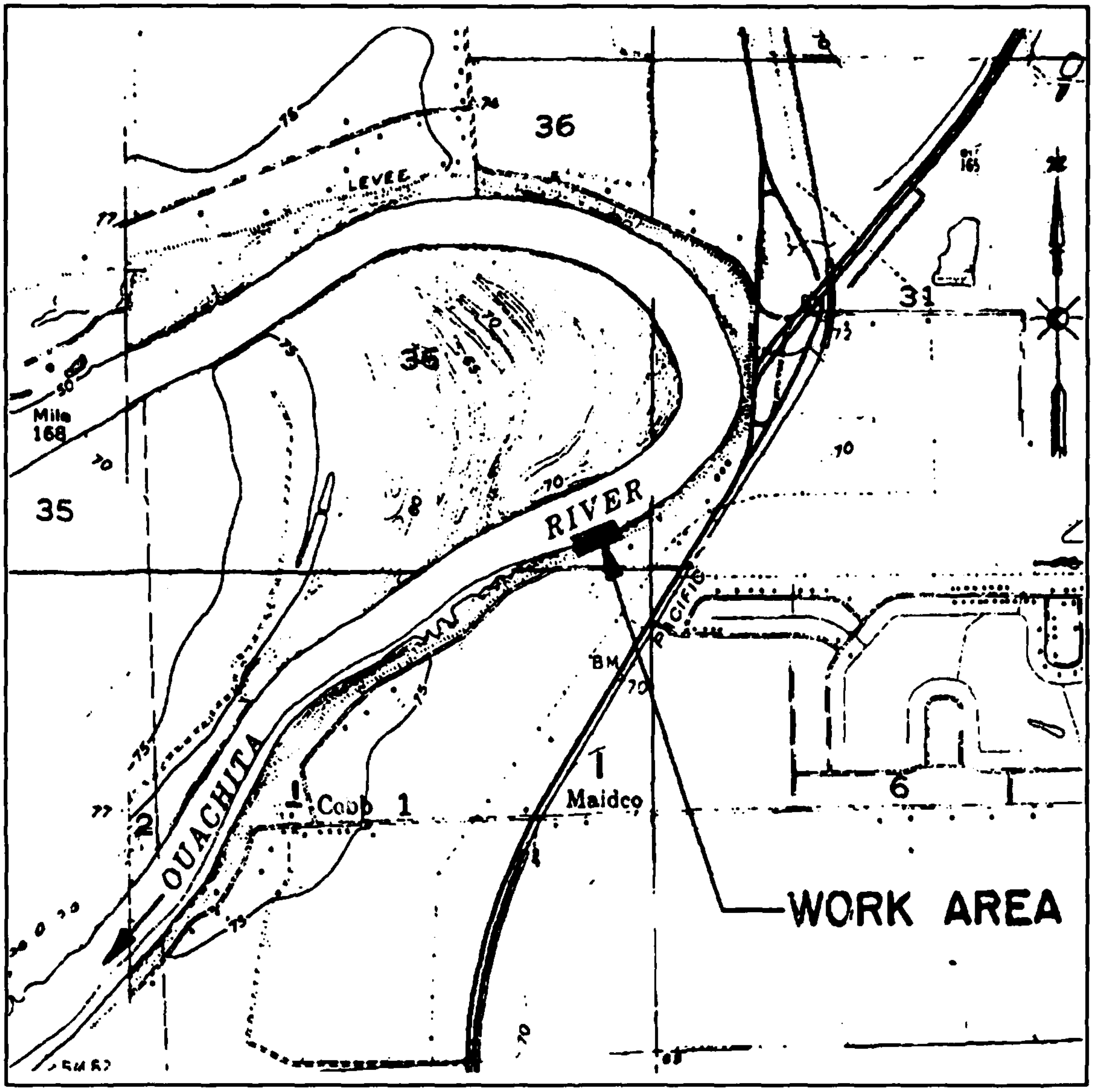

Figure 1. Site location

Slide Assessment

Geology. The area is located in the western part of the Mississippi Alluvial Valley in a region known as the Ouachita lowland. Fleetwood (1969) presented a detailed study of the area geology. The alluvial deposits in the area consist of an upper fine grained unit called the topstratum and a lower coarse grained unit called the substratum. The alluvial topstratum deposits of the Ouachita River Valley are classified according to depositional environment. The topstratum at the Rilla site, located 9 miles downstream of Monroe, Louisiana, at approximate River Mile 166 on the Quachita River, consists of a 2- to 5-ft thick sequence of natural levee silt (ML) which overlies a

20- to 25-ft thick deposit of medium to very stiff backswamp clay (CL and CH). Natural levee deposits form broad low ridges that flank both sides of the river and are a result of overbank deposition during periods of flooding. Backswamp deposits consist of fine grained sediments laid down in broad shallow basins within the floodplain during major periods of stream flooding. The topstratum deposits rest directly on a 40- to 50-ft thick layer of highly erodible substratum sand (SP and SM) with a 2- to 5-ft gravel strata at the base. These alluvial deposits in turn lie directly on the Tertiary age material that consists of a hard, dark gray clay (CH).

Exploration program. The investigation of the soil conditions at the site consisted of three undisturbed borings taken in September 1983 and two undisturbed borings taken in June 1988. The borings indicated a silt layer of the natural levee deposit and a backswamp clay deposit consisting of a layer of very stiff clay (CL) overlying a medium to very stiff clay (CH). The generalized soil and slide profile is presented in Figure 2. The borings which were made through the existing levee are shown in plan on Figure 3.

Laboratory testing. Laboratory testing consisted of water content determinations, Atterberg limit tests, and unconfined compression (UC) tests. The UC tests indicated the shear strengths range from a cohesion of 250 psf to 780 psf with an average value of 500 psf. The water contents varied from a low of 21 to a high of 42. It should be noted that the slickensided nature of the backswamp clay helps to contribute to the lower UC values.

Conditions contributing to slide. The site investigations in 1983 and 1988, cross sections of the area taken on regular intervals and familiarity with the construction problems associated with the 1984-85 repair led to the conclusions that the initial triggering mechanism for the slide was the steepening of the river bank due to erosion of the silty sand substratum. This in turn caused sliding to occur in the backswamp clays. The initial attempt to stabilize the slide using the stone dike and sand berm was to consist of building a stone dike to elevation 62 and rebuilding the slope to a 1V on 3H with sand fill. The sand fill would be protected from scour with engineering fabric and riprap. In November 1984, high water conditions existed at the construction site. The contractor chose to place the sand fill prior to completion of the stone dike. The weight of the additional sand fill caused the slide to experience further movement in the backswamp clay. The contract was modified to ensure the stone dike was completed before any more sand fill was placed. The modification also allowed placement of the riprap in other than dry conditions which was precluded in the original contract. The work was completed in January 1985 and the section continued to experience some movement. This movement was assumed to be occurring along a remolded slip plane that existed in the backswamp clay. The movement was more than strain cracking, but did not seem to be endangering the levee. In the summer of 1988, the movement had reached the point that it was considered to be an endangerment to the levee. It was also clear at this time that the slip plane was located in the backswamp clay and that further attempts to stabilize the slide must plan for this condition.

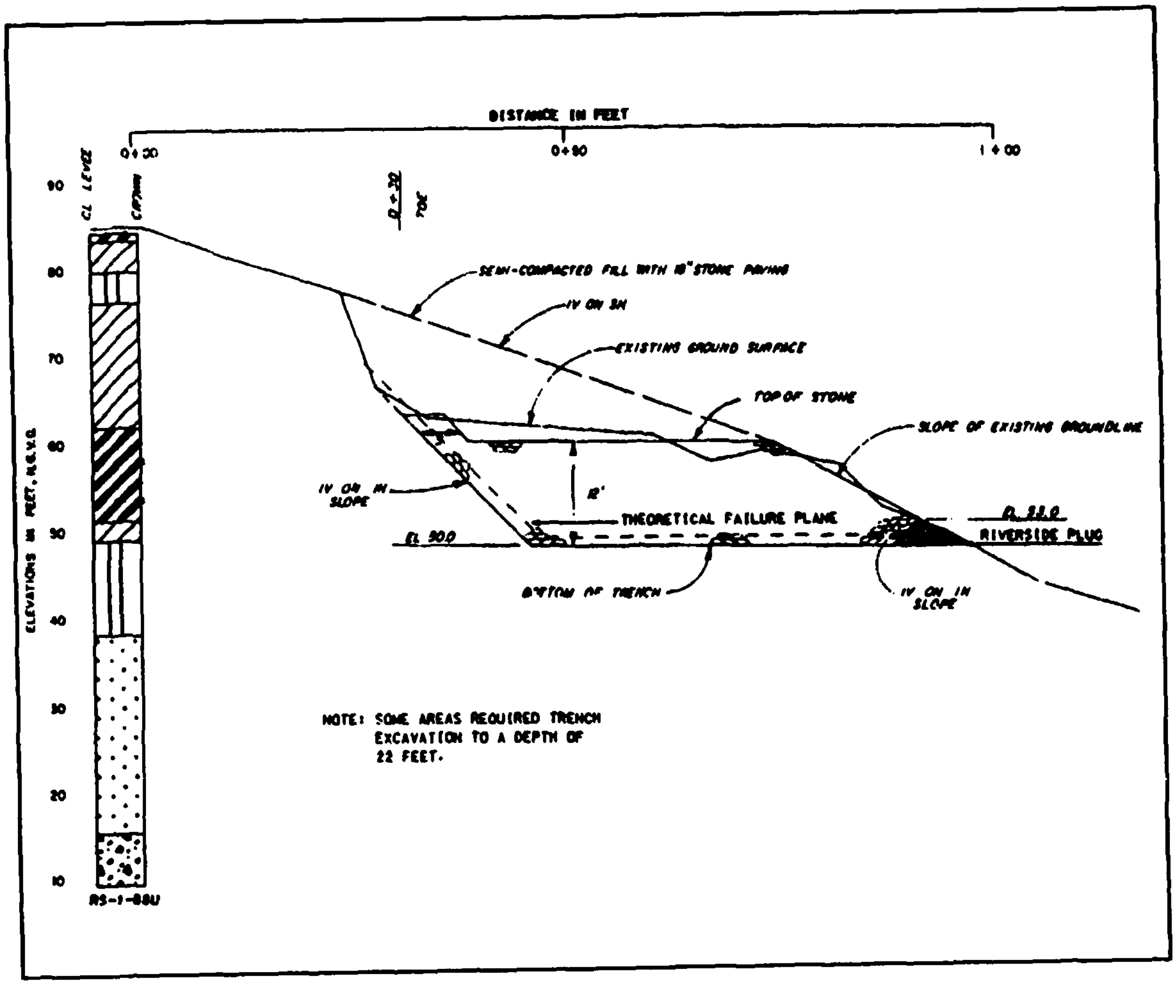

Figure 2. Typical section of slide area

Remedial Measures

The selection of stone-fill trenches as the method of stabilization at Rilla was based on considerations that the use of a berm large enough to stabilize the slope would be both a hinderance to navigation as well as be extremely expensive because of the large quantity of stone required. Slope flattening was not a feasible alternative because the levee could not be set back further. This left only some form of in situ stabilization as an acceptable alternative. An additional consideration was the emergency nature of the work and the short time frame allowed for plans and specifications. A method that could use readily available equipment and be bid by local contractors was highly desirable. A first consideration was to use trenches to stabilize the slide. Because the slide area was already protected with 18 in. of stone paving, it was decided to use this stone to fill the trenches. Quarry-run stone would further provide a material whose strength was less susceptible to degradation due to contamination from clay and silt fines. Quarry-run stone would be

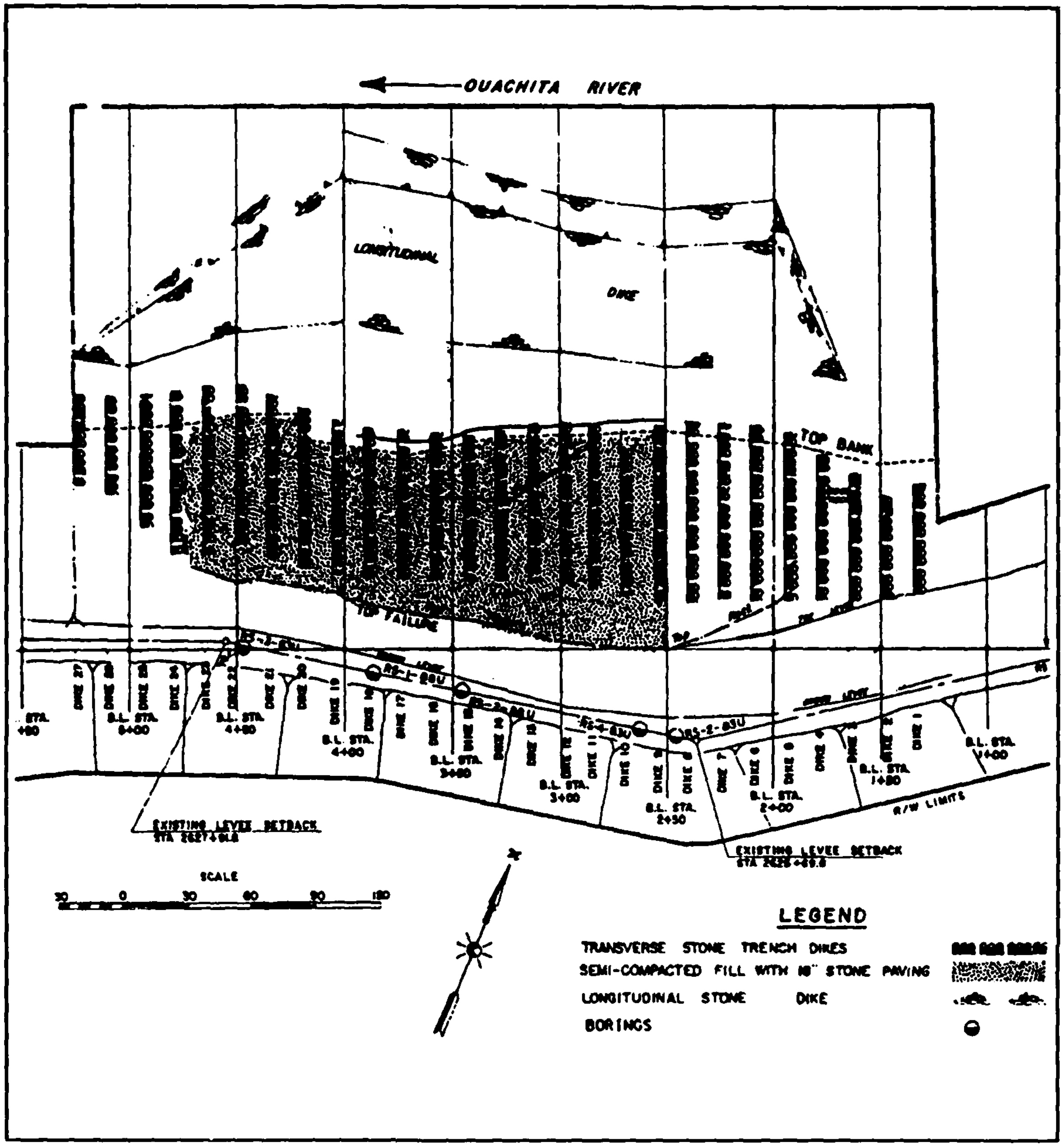

Figure 3. Site plan

much larger and more angular than rounded, pit-run washed gravel. The concept of stone-fill trenches is similar to stone column replacement and is applicable where digging of the trenches is within the capability of the digging equipment and conditions. In this case the maximum trench depth required to extend the trench below the bottom of the backswamp clay was 22 ft. This depth is below the theoretical depth an open trench will stand in this type of material. The depth of cut could be reduced by leveling or removing material from the top of the slide zone where possible. However, it was decided to minimize the double handling of material and modify the construction technique by only opening a small portion of the trench before backfilling.

Design of remedial measures consisted of detailed analyses of the existing conditions to determine a strength acting upon the inferred slip surface at the base of the backswamp clay. Figure 2 represents the cross section of the slide, with the boring log plot, and the location of the inferred slip surface which was used in the analyses.

A computer program utilizing a wedge method was selected to analyze the slip surface, assuming the sliding was to be in a state of limiting equilibrium. This program was written by Cheek and Hall (1975). Even though no slope indicator data were available to define the slip surface, assuming the slip plane at the bottom of the clay layer was considered conservative. These analyses also assumed a phreatic profile of 5 ft below the ground surface.

Drained strength parameters of $\phi' = 16°$ $c' = 0$ psf were backfigured as an average strength acting on the slip surface producing a safety factor of unity.

Once the strength acting on the slip surface was determined, it was necessary to determine the average strength required to provide a factor of safety of 1.25 for the repaired slope. Analyses indicated that an average drained shear strength of $\phi' = 20°$ and $c' = o$ psf would provide the shear strength required to yield the required factor of safety.

It was decided that the existing riprap on the slope would be used in the trenches. The shear strength was estimated for this material as $\phi' = 40°$ and $c' = 0$ psf. The trenches were to be excavated to the bottom of the backswamp clay and filled with a minimum of 12 ft of rock. Semicompacted fill would be placed on top of the repair to restore the section back to the original section.

Assuming that a 2.5-ft-wide bucket would be used to excavate the trenches, and with the shear strength parameters for the slip surface and stone-fill known, the next step was to determine a trench spacing that would provide the composite shear strength required to raise the computed factor of safety to 1.25. To determine the trench spacing, an equation was developed using a weighted average principle. In the equation, the ϕ' of the existing soil mass is multiplied by the distance between trenches. This term is then added to the ϕ' of the trench backfill material multiplied by the width of the trench. These terms are equal to the ϕ' of the composite section multiplied by the total distance between the trench plus the width of the trench. The Sills/Fleming equation used is as follows:

$$\phi_r' \times S + \phi_f' \times W = \phi_a' (S + W) \tag{1}$$

where

ϕ_r' = the average ϕ' acting on the slide mass
S = distance between trenches
ϕ_f' = ϕ' of the trench backfill material

W = width of trench
ϕ_a' = the average ϕ' of the composite section

Since the trench spacing is generally measured center to center

$$S + W = T_s \tag{2}$$

where

T_s = trench spacing measured center to center

Therefore equation (1) can be rewritten

$$\phi_r' \times S + \phi_f' \times W = \phi_a' \times T_s \tag{3}$$

Solving equation 3 it was determined that a center to center trench spacing of 15 ft would provide the required average strength parameters of $\phi' = 20°$ and $c' = 0$ psf. It should be noted that since arching of the soil should occur between the stone-fill trenches the actual factor of safety should be greater than 1.25. A typical design section is shown in Figure 4.

Construction

Construction of the stone-fill trench stabilization for the U.S. Army Engineer District, Vicksburg, in December 1988 was accomplished by contract. The contract required that 27 transverse stone-fill trenches (Figure 3) be constructed beginning downstream and proceeding upstream. The work area was within the Columbia Lock and Dam Pool which maintained a minimum river of 52 ft NGVD. Since the trenches were required to be excavated to elevation 50, a soil plug was left in place on the riverside of the trench, Figure 2. The trenches were constructed beginning at the riverside plug and proceeding landward toward the levee toe.

Prior to trench construction, existing bank paving was removed along the trench where excavated material would be placed. Trench excavation was performed using a tracked backhoe. The trenches were excavated to full depth. Excavated material was placed downstream of the trench in a manner not to overload the bank. This also ensured the excavated material was placed on a stabilized section and was not mixed with the surface stone. As the backhoe moved upslope, the existing bank paving was placed in the trench as backfill. This allowed the backhoe to both excavate and backfill the trench. The contractor was required to have less than 5 ft of unfilled trench open at any time. The backhoe moved upslope until the vertical scarp was encountered, the trench was backfilled, the backhoe turned 180° and finished the trench, digging from the top of the scarp to the previously complete portion of the trench. Figure 5 is a photo of the slide area.

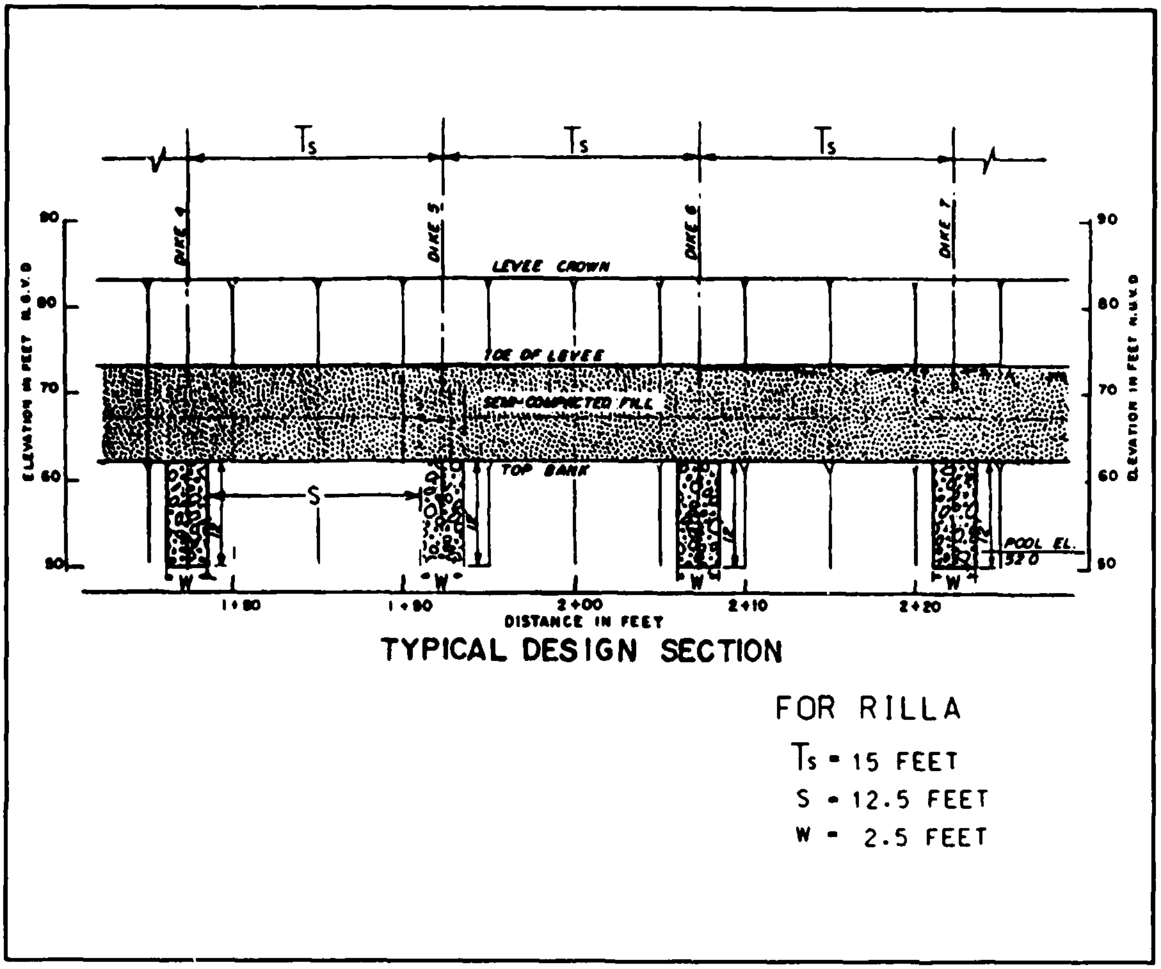

Figure 4. Typical design section

This procedure was repeated for each of the 27 trenches. The soil excavated from the trenches was used to backfill the slide area to the original section. Finally, 18 in. of bank paving was added to the repair area. Figure 6 is a photo of the completed work. The construction of the repair was completed in only 12 days with a final total cost of less than $100,000.

Monitoring and Performance

After completion of this repair in December 1988, there has been no visual movement of this slide. During the spring of 1991, the Ouachita River reached record stages in this area. After flood stages rapidly decreased, the repaired area was still intact.

Figure 5. Slide area

Conclusions

The stone-fill trench method of slide stabilization has a definite application in appropriate situations. These situations are shallow slides in soil masses with enough integrity to remain stable when trenches are excavated below the slip plane with near-vertical slide slopes. This application has several definite advantages when compared with other methods of stabilization. Some of the merits are relatively low construction costs, moderate level of design complexity, and the ability to be performed with conventional, readily available, construction equipment.

The stone-fill trench method of stabilization met the basic requirements of the project, which was to stop the sliding of the unstable soil mass and to restore the levee section to pre-slide conditions.

The stabilization of this slide using stone-fill trenches serves as a prototype for other slide repairs. It provides information that can be used to evaluate and, hopefully, refine this method into a more economical and technically

Figure 6. Completed repair

viable stabilization technique. One needed future refinement would be a study of the arching effect of the soil between the stone-fill trenches.

Acknowledgment

The authors would like to express their thanks to the U.S. Army Corps of Engineers, Vicksburg District, for undertaking this method of stabilization. The authors would also like to thank Messrs. Todd Hill, McKinley Harris, and Eddie Stewart for performing analyses and preparing figures, Mr. Stephen Lee for assisting in preparation of the geology, and Mrs. Sue Richardson and Mrs. Jaudon McKay for typing this paper.

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Units

1 ft = 0.305m
1 in. = 0.0254m
1 psf = 0.0479 kn/m^2

Lime-Fly Ash Injection of Levees

Jerry A. Holloway
Kansas City District

Introduction

The particular levee discussed in this case history was constructed as a part of Unit L-246 of the Missouri River Levee System near Brunswick, Missouri River Levee System near Brunswick, Missouri by the Kansas City District Corps of Engineers. It is located at the confluence of the Lower Chariton River with the Missouri River and was constructed between 1965 and 1971. In 1983 a slide failure was noticed in the levee on the right bank of the Chariton River. Within 24 months nine slides occurred in this section of levee affecting 1,300 lin ft of levee. This condition resulted in a drastically reduced level of flood protection provided by the L-246 levee unit. The unsupported levee slope was only 11-13 ft in height and had not been submitted to flood waters during that time period. Further investigations revealed that approximately 13,600 lin ft of the levee had been constructed of highly expansive montmorillonite-type clays. It was placed by utilizing the uncompacted fill option of the specifications. The uncompacted fill option allowed the material to be placed in a saturated condition and was placed in approximately horizontal layers not exceeding 3 ft in uncompacted thickness, being spread, distributed, and otherwise manipulated during placement to the extent that individual loads of material deposited on the fill did not remain intact, eliminating large open voids in the fill.

Test Sections

After reviewing the different possible methods for repairing the levee, it was apparent that lime/fly ash slurry injection could offer a considerable cost savings over the more conventional method of rebuilding the levee of non-expansive material or mechanically incorporating a stabilizing agent into the levee embankment.

Treating a failed embankment with lime/fly ash slurry is accomplished by injecting the lime/fly ash at high pressures into the soil mass. As the slurry is injected, it is forced along weak planes such as fractures and voids within the embankment. The presence of fractures and voids is particularly notable in embankments composed of expansive soil. The desiccation patterns found in the soil mass lend themselves well to the injection process.

Once the lime/fly ash is installed it reacts as a pozzolan and gains strength while reacting with the soil at the contact areas.

The injection results in an increase in the strength of the overall soil mass as well as an increase in the unconfined compressive strength. By filling the cracks and fissures of the embankment with a cementitious material the lime/fly ash injection not only increases the strength of the embankment but also prevents water intrusion through previously existing cracks. Lime/fly ash injection was further investigated by constructing some test sections in some failed areas of the levee. In November of 1984, three different areas of the L-246 levee were test-injected. One of the areas was a section of levee in which a 200 ft long slide existed. It was decided that 100 ft of the failed reach was to be injected and the other 100 ft left as a control section. The entire length of the slide was pushed back into place without any compaction and the slope was redressed. The 100-ft test section was injected twice with lime/fly ash slurry to a depth of 10 ft. A second area of the Chariton River Levee that had not shown any signs of weakening was injected. The third area was injected at the cut-off lake levee on Palmer Creek, where a vertical 18-in. scarp had formed on the lakeside edge of the levee crown. This area was over 100 ft in length. The test sections were then monitored during the next few months. During the period of time that the test sections were monitored, the (untreated control section experienced) a slide failure; however; none of the injected areas experienced any further disturbance and have since remained intact.

Practical Application

Based upon the successful results obtained from the test section and the potential cost savings, and linked with the strong desire to have continuous flood protection during the rehabilitation, it was determined that the entire lengths of both reaches of the right bank of the Chariton River Levee that was composed of uncompacted expansive clay should be injected with lime/fly ash slurry.

The Woodbine Division of Hayward Baker was the successful contractor and the work began in September of 1987 and completed in 1988. The scope of work involved 14,950 lin ft of levee to be repaired and/or stabilized. Each side of the levee was injected from toe to top. Areas where slides had previously occurred were redressed, pushed into place, and compacted prior to the injection.

The lime utilized for the project was a commercially available quicklime (CaO) that was slaked into hydrated lime (Ca(OH)2 in a lime slaking tank after delivery to the jobsite. Fly ash was obtained from a local coal-fired generator and was a class "C" which contained 20 percent to 25 percent CaO. The dry fly ash was added into a lime slurry through a jet slurry mixing valve. A ratio of one part lime to 3 parts fly ash was mixed in the range of 4 pounds lime/fly ash per gallon of water. It was injected within a pressure range of 50 to 70 psi.

As the slurry was manufactured it was transported down the levee to the point of injection (a maximum of three miles) through a series of pumps and hoses. At the injection area the slurry was received into a deaeration pod where the specific gravity was checked again before the material was pumped to the injection units. The injection units consisted of dozers with a fork lift assembly mounted on front. Each fork lift "mast" contained four injection rods spaced 5 ft on center. The rods were injected into the ground simultaneously by the hydraulic force of the forklift mast. As the rod entered the soil, the tight clay formed a seal around the pipe which forced the slurry flowing out of the pre-drilled holes in the bottom of the pipe to flow into the soil around the pipe. The slurry flowed until refusal (i.e. the material was visible at the surface running freely from cracks in the soil and/or previous injection holes).

At refusal the rods were forced another 12 to 18 in. into the soil and the pattern was repeated until the proper depth was achieved (5 ft and 10 ft). Before completely extruding the injection rod from the hole, the pump was left on to fill the hole with slurry, thus not leaving an empty hole in the face of the slope.

Injections were accomplished in two phases (i.e., primaries and secondaries) and within a pressure range of 50 to 70 psi. The average minimum quantity of lime and fly ash being used is 1.25 lb/cu ft for primaries and 0.75 lb/cu ft for secondaries. These are based on the total area to be stabilized.

However the slurry either shrank or a phase separation occurred, where the solid constituents of the slurry settle out and the bleed water continued to penetrate the embankment during the curing process leaving a partially empty hole in the face of the slope These holes when first found and measured varied in depth from 3 to 10 ft with the majority being 3 to 4 ft in depth.

Investigations and discussions concerning these voids resulted in several different procedures being implemented, including using heavier mix with the second injection (approximately 6 lb of lime and fly ash per gallon of water), reversing the injection sequence to begin injecting at the crown and then proceeding down the slope, and/or keeping the slurry pumping until just after the injection rods are withdrawn from the ground. These efforts helped, but did not eliminate all the voids, and in the upper third of the levee slope some of the voids were still open to a depth of 2 to 3 ft. In the lower two-thirds of the slope, the voids were less than a foot deep, and in many holes the voids were

completely filled. A harrow was then used to work up the surface of the slope, thus filling any remaining voids.

During subsequent investigations, it was noted that the voids were not detectable and that the overall objective of strengthening of the total embankment had been achieved. An alternative method that would eliminate this problem would be to strip the top 4 to 6 in., mix it with waste slurry and replace at the end of the project. Whatever method might be used for future contracts, the specifications should include a description of final conditions.

Lime Slurry Information

As water is added and mixed with the quicklime in the lime slaking tank, the Specific Gravity (SG) of the resulting slurry is continually monitored until it gets in range of approximately 1.06 to 1.12. The temperature and the SG are then recorded on the daily production report. The pounds of lime per gallon of slurry (PL/GS) for the produced lime slurry can then be obtained. The gallons of slurry mixed is found by measuring in inches the distance from the top of the slaking tank to the level of the slurry tank.

Lime-Fly Ash Slurry Information

The SG of the slurry in the mixing tank is monitored nearly constantly. This is necessary due to the inaccuracy of the jetting process for introducing the fly ash to the lime slurry. The SG readings are entered on the Daily Production Report, and are then averaged for the shift. From the average SG of the lime-fly ash slurry, the average pounds of lime-fly ash per gallon of slurry can also be calculated using the formula shown.

The method of production of the lime-fly ash slurry as described above is far from an exact means. Using this method it is impractical and nearly impossible to get the slurry to exactly one part lime to three parts fly ash, as is called for in the contract specifications.

SG measurements of both the lime slurry and the lime-fly ash slurry are extremely critical and need to be as accurate as possible. Slight errors in SG measurements of the slurry will give very inaccurate indications of the amount of lime and/or fly ash actually injected.

At the beginning of the project, baroid mud scales were used to obtain SG's of the slurry. Although faster and more durable than other means such as hydrometers, it was felt the results were not accurate enough. Hydrometers capable of measuring to the nearest 0.0001 were then used exclusively on the lime slurry, and are being used as a check in conjunction with the baroid scales, on the lime-fly ash slurry.

At the present time it is too early to tell if this process is going to be considered a success or a failure. It is felt that any recommendations be withheld until a number of years after the completion of the project to see if any problems develop.

Levees Constructed on Soft Soils Using High Strength Geotextiles

Philip J. Napolitano
New Orleans District

Why Geotextile Reinforcement

The design and construction of embankments on soft foundations has always represented a challenge to engineers in the New Orleans District and elsewhere. It is probably as much art as scientific engineering. Multi-lift construction and large berms have frequently been the selected approach, often resulting in costly, time-consuming construction to achieve the desired final grade.

Cost

The design engineer must take into consideration all the geotechnical data available, the embankment construction materials, the availability of suitable borrow, construction techniques, location constraints, environmental factors, and, of course, time and costs. These considerations are not necessarily all-inclusive, but may vary with each individual project. Generally speaking, embankments on soft foundations result in large cross sections that include stability berms. The use of geotextiles (fabric or plastics) may reduce the size of these embankments. The cost (materials and installation) of the appropriate geotextile must be offset by the reduced fill material needed, the smaller right of way required (real estate costs), and perhaps a quicker completion time. Geotextiles are generally not cost effective if some or all of the following are applicable: availability of inexpensive backfill material, enlargement of an existing embankment, low real estate costs, and few time constraints.

Real estate

On some occasions, the use of a more costly unit design may ultimately be the selected alternative if other constraints are present. As an example, a

smaller, but more expensive concrete floodwall may be the desired solution if the less expensive, but larger, earthen embankment simply will not fit within the available right of way, and obtaining the necessary additional real estate is simply not an option. The use of geotextiles in this instance could be more cost effective than the expensive concrete structure. Related to this situation is the enlargement of an existing embankment. Raising the design grade may increase the required cross section to an unacceptable base width, given the available right of way. The use of geotextiles, although more expensive than the additional backfill material, offers the more cost effective solution when compared with the concrete structure, or the consequences of not raising the embankment.

Time

In many instances, time is a prime consideration in the design and construction of an earthen embankment on soft foundations. This is a result of the nature of soft clays, in that they consolidate when loaded, and take "long periods of time" to complete this consolidation. As a result, the designer must early on choose between "multi-lift stage construction" with many years between lifts to allow for consolidation and the resultant gain in strength, or a larger single lift constructed to final grade. The use of geotextiles generally will not reduce the ultimate consolidation, but will allow construction to a higher elevation, thus permitting a quicker construction to the ultimate design grade.

Design Considerations

It is the intent of this paper to present a very brief and highly simplified, general overview of the design considerations necessary to design reinforced embankments on soft foundations. As this is a relatively new application in geotechnical engineering, little definitive data and research are currently available, and some controversy exists on what theories apply and how to use them. A designer must thoroughly study the conditions applicable to a site specific project, obtain the necessary data, and apply appropriate engineering theories and judgment. This paper attempts to provide a starting point for that process.

Conditions

Global stability. One of the first considerations in the design of embankments on soft foundations is to consider the overall or "global" stability of the embankment. This includes a "rotational" or shear failure within the embankment, and possible a "deep failure" into the foundation. Global stability must also include a check of a potential bearing capacity failure.

Shear failure. A potential shear failure within the embankment or foundation is usually checked by some sort of rotational stability analysis. Most of these analyses are well documented. Circular failure surfaces are generally assumed in homogeneous materials. Wedges or planes are more commonly utilized in stratified foundations, especially in those containing weak planes. Please refer to Figure 1. Because of the very soft and stratified nature of the foundation conditions in the New Orleans District (NOD), a wedge analysis (the LMVD Method of Planes) is utilized.

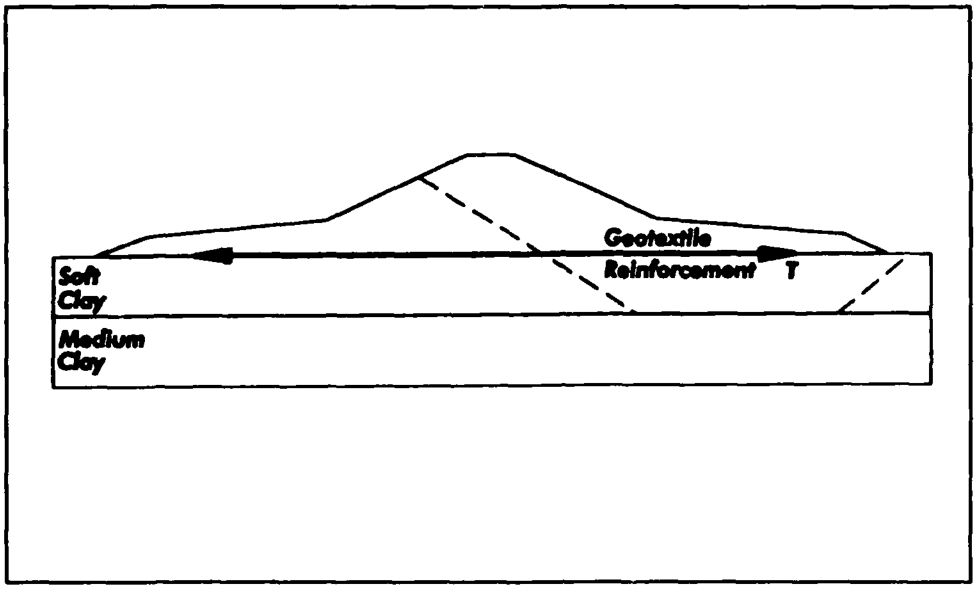

Figure 1. Global stability wedge analysis

The Factor of Safety (FS) is computed by dividing the summation of Resisting Forces by the summation of Driving Forces.

$$FS = \frac{Ra + Rb + Rp}{Da - Dp}$$

where

Ra = Resisting active force (along active wedge)
Rb = Resisting base force (along neutral block)
Rp = Resisting passive force (along passive wedge)
Da = Driving active force (active wedge)
Dp = Driving passive force (passive wedge)

Figure 2 illustrates the assumptions in geometry and forces, and the resulting equations of the LMVD Method of Planes. A Factor of Safety of about 1.3 is generally required. If the summation of resisting forces is not large enough to provide the appropriate FS, either stability berms may be added or a geotextile may be utilized to provide the additional resistance. This

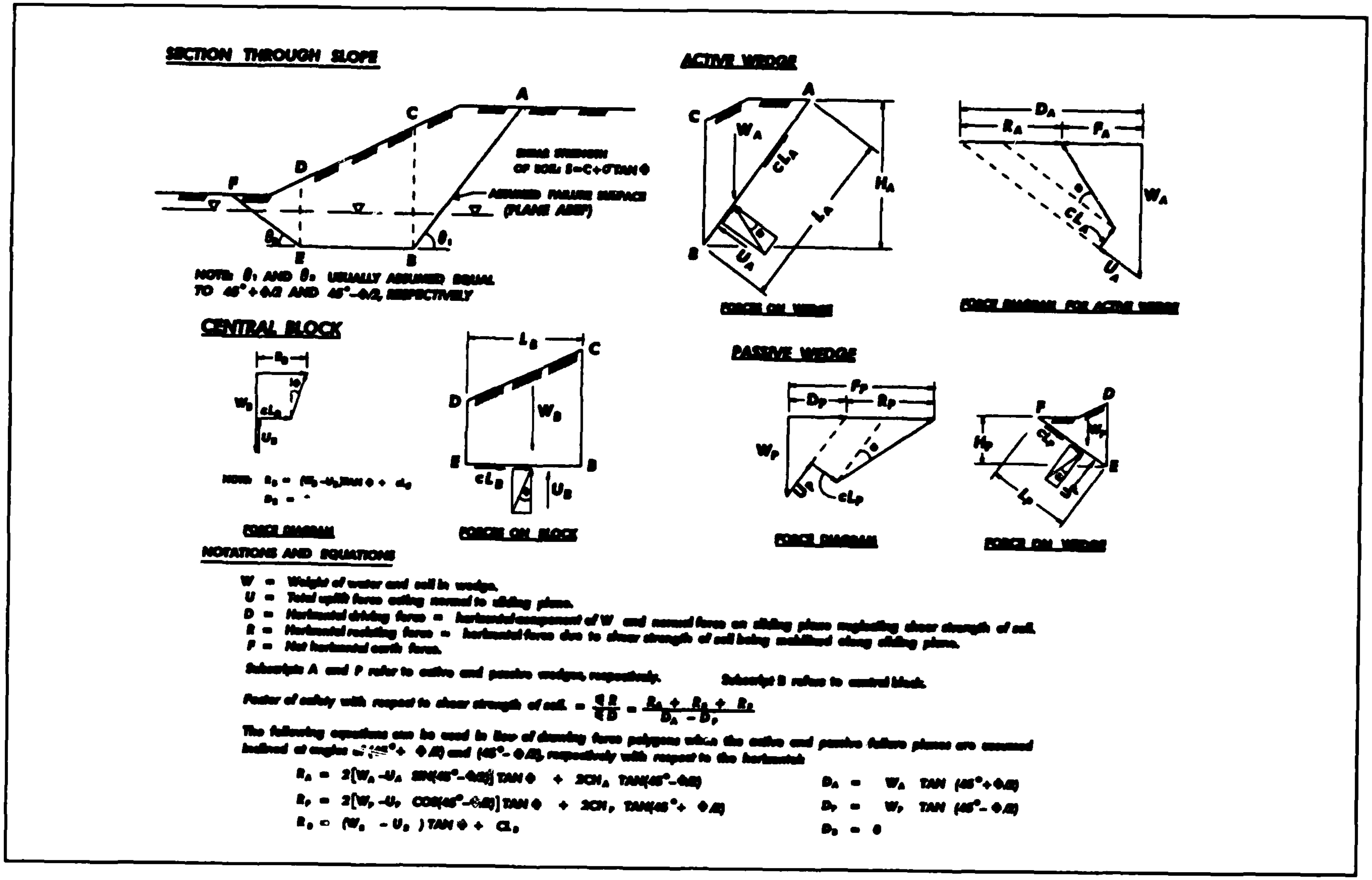

Figure 2. Assumptions in geometry and forces, LMVD Method of Planes equations

resistance is obtained from the tensile strength of the fabric embedded in the backfill material (or in the foundation). The required tension (T) is calculated by subtracting the summation of Resisting Forces from the summation of Driving Forces × FS.

$$T = FS\ (Da - Dp) - (Ra + Rb + Rp)$$

Since the above described method makes no provisions for the location of the reinforcing layer (geotextile), i.e., the T value is the same no matter where the fabric is located within the embankment section or foundation, another analysis is performed to provide for position. The New Orleans District (NOD) uses the Spencer analysis to "fine tune" the value of T obtained above. See Figure 3. This analysis (one of the options of the UTEXAS2 program) calculates the FS utilizing a "center of rotation." Therefore, the horizontal locations of all forces affect the resulting design.

$$T = \frac{FS\ (D - R)}{Y}$$

where

T = Geotextile tensile force
D = Driving moments
R = Resisting moments
Y = Moment arm for tensile force (T)

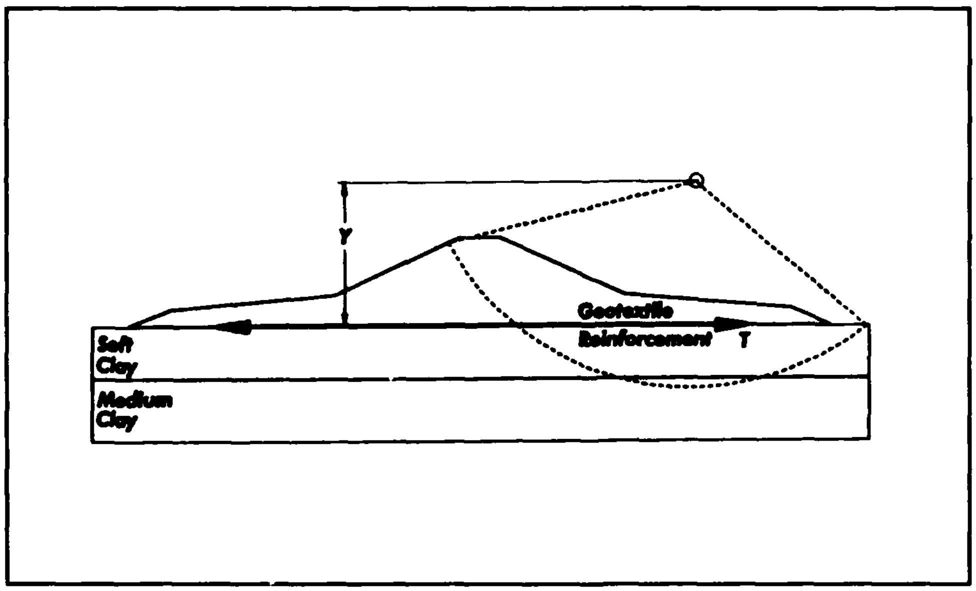

Figure 3. Global stability Spencer analysis

The Spencer analysis is first run without any tensile reinforcement to see if the analysis "approximates" the results of the LMVD Method of Planes

(wedge analysis). A geotextile layer is then added at the approximate location, usually the base, assumed in the wedge analysis. The FS and T force is then recomputed with the appropriate Y (moment arm). These are compared to the values obtained with the wedge method. The final T value is selected based on engineering judgment. A geotextile is then chosen with the appropriate stress-strain curve and tensile strength. To allow for long-term creep, an ultimate tensile strength of about twice the required T value usually results. (This is based on an allowable strain of about 5 percent.) The "pullout" or embedment length of the fabric must be calculated to ensure that the fabric is properly embedded in the embankment or foundation. Little is currently known about this aspect of design. Design values and procedures will be discussed in subsequent paragraphs.

Bearing capacity. In designing any structure on a soft foundation, bearing capacity must be checked. This includes an embankment reinforced with geofabric. The geofabric, when properly anchored at the ends, tends to "span" the embankment, preventing small localized bearing capacity failures and causing the embankment to act less like a "flexible" structure. If the reinforced embankment is placed on a soft, very deep and homogeneous foundation with no increase in strength with depth, classical methods of computation are well documented in the literature. However, soft foundations may not extend "very deep," and generally show an increase in strength with depth. Further, there may be a relatively hard layer underlying the soft soil, and within the potential failure zone of the embankment. This may be rather common, given the wide base of embankments on soft foundations. A method to deal with this situation was contained in a paper presented by R. K. Rowe (Rowe and Soderman 1987) at the First Geosynthetics Research Institute Seminar at Drexel University in 1987. The method generally presented a number of graphs relating the various design and geometry parameters to the bearing capacity factor N_c. Please refer to Figure 4 for basic parameter definitions and to Figure 5 for a simplified example of some of the graphs. Once the value of N_c is known, the allowable bearing capacity may be calculated. A factor of safety of about 1.25 to 1.5 is used by the NOD. It should be pointed out that the presence of a hard layer within the influence of the embankment, and the case of an increase in shear strength with depth, will result in higher bearing capacity, and therefore, higher embankments.

Elastic deformation. This condition refers to the stretching or elongation of the fabric under the load of the embankment that induces the "T" force. The elastic deformation of the fabric then allows the embankment to move in a lateral direction and thus vertically, as illustrated in Figure 6. This movement by the embankment must be controlled (by limiting fabric strains), as excessive movement will cause unwanted settlement, possible severe cracking of the fill material, and ultimately, failure. The amount of allowable strain in the fabric will be a function of the embankment backfill material, the tolerable settlement, the stress/strain compatibility of the soil and fabric, and constraints within the rights of way, etc. Some values of maximum strain suggested in recent studies approximate 10 percent. Thus, when calculating the "T" value

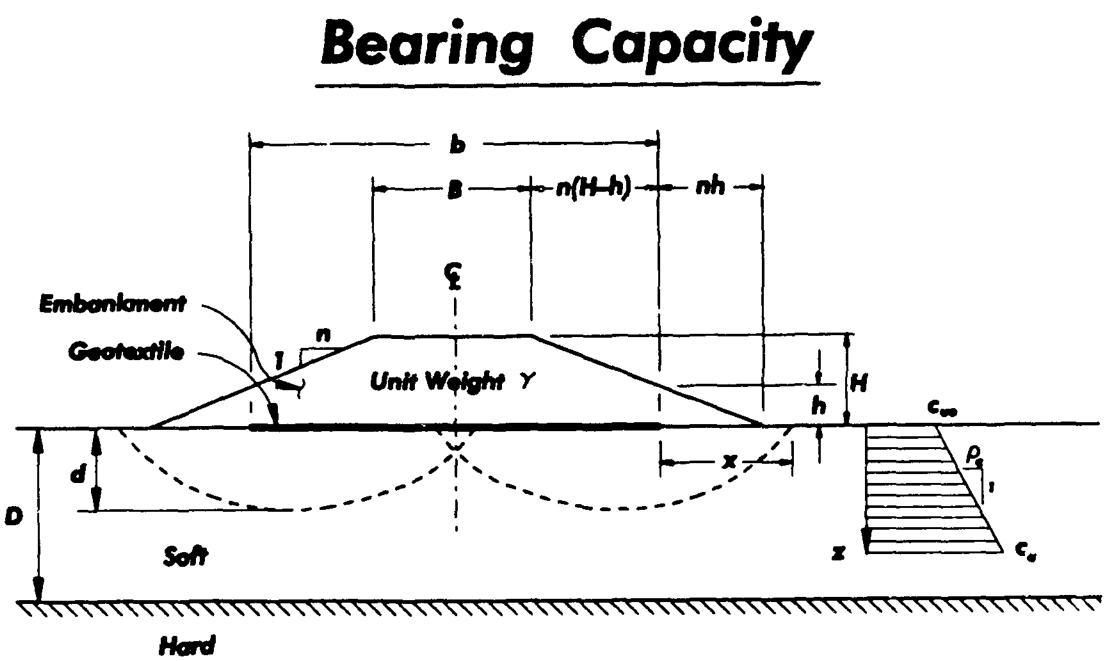

Figure 4. Definition of variables used to estimate H for reinforced embankment

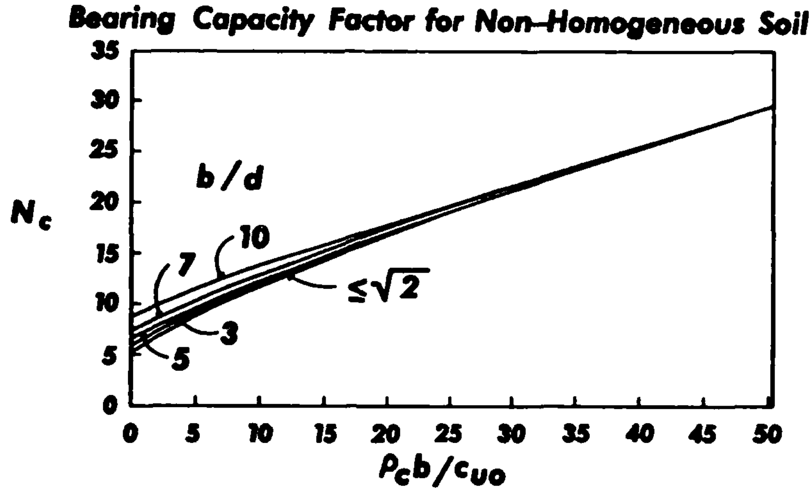

Figure 5. Bearing capacity factor for non-homogeneous soil

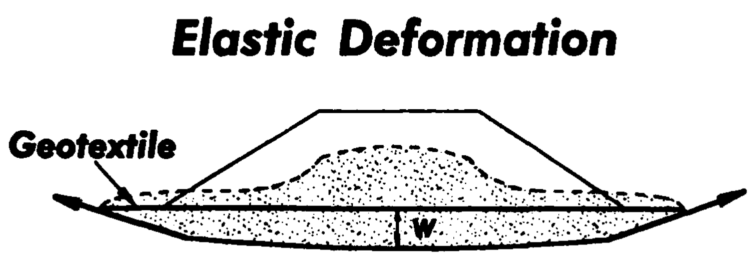

Figure 6. Elastic deformation

required for global stability, as discussed above, the ultimate strength of the fabric will be much higher, based on its stress/strain curve--Modulus of Elasticity.

$$E = \frac{T}{E}$$

where

E = Modulus of elasticity
T = Tensile stress in fabric
E = Strain

Please note that creep, as previously discussed above, must also be taken into consideration. As a result, the ultimate strength of the fabric selected may well be governed by its modulus of elasticity because of the limiting elastic deformation and creep requirements (a value of 5 percent strain for creep is often used in the New Orleans District), and not by tensile values required for global stability or bearing capacity.

Lateral spreading. Figure 7 illustrates a possible failure mechanism related to lateral spreading. In this situation, a crack may develop in the crown of the embankment when the backfill material spreads laterally away from the centerline, moving above the geotextile at the soil/fabric interface. This possible failure mechanism is a function of the embankment height, the shear strength of the backfill material, resistance along the soil/fabric interface, and the length of the embedded fabric. It has been suggested that this "situation only becomes severe for steep slopes and very smooth geosynthetic surfaces." (Koerner, Hwu, and Wayne 1987) The potential for failure may be approximated by the following equation.

$$Pa = \tau L$$

where

Pa = Active force
τ = Resistance along soil/fabric interface
L = Embedded fabric length

The active force (Pa) is calculated using well-known geotechnical equations for lateral earth pressure theory and the corresponding earth pressure

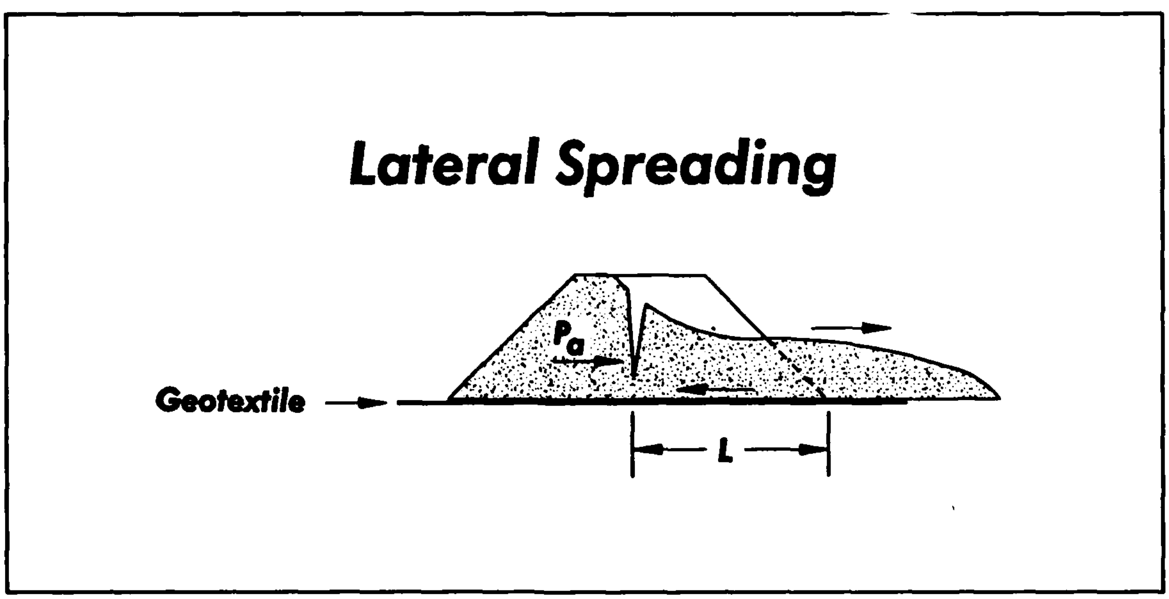

Figure 7. Lateral spreading

coefficients. The resistance (τ) is calculated from equations that "estimate" frictional resistance along a surface. This resistance is generally a function of overburden pressure and the shear strength of the overburden material, depending on whether it is cohesive and/or granular. This subject will be discussed subsequently under "pullout considerations."

Settlement. Consolidation (settlement) probably would not be considered a "failure mechanism." However, it is an important design consideration. Obviously, if a minimum embankment grade is required, consolidation of a soft foundation resulting from the embankment weight must be estimated. Consolidation theory and the applicable equations are well documented in geotechnical literature. The important point to be noted here is that the presence of geofabric reinforcement within the embankment does not alter the ultimate consolidation of the foundation below. The total weight of the backfill material is still acting upon the subsurface. The presence of the geofabric may "span" some localized soft areas, and thus minimize "local" consolidation. This redistribution of stresses may also alter the time-settlement curve. However, since the total weight remains unchanged regardless of the presence of the geofabric, the ultimate consolidation is the same. (Lateral spreading, which affects settlement, may be reduced.)

Pullout considerations. After the "T" value is determined, based on global stability, the embedment length "L" must be calculated. Figure 8 illustrates the basic geometry. The value "L" is the length of fabric required to resist the tensile force "T" developed in the fabric by the weight of the embankment and the characteristics of the subsurface. The equation utilized to estimate "L" is:

$$L = \frac{T}{\tau_a + \tau_b}$$

where

L = Required embedment length
T = Tensile force in geotextile
τ_a = Resistance above geotextile at interface
τ_b = Resistance below geotextile at interface

The value of τ is a function of the overburden, the shear strength of the backfill material, and roughness of the fabric, and may further vary with water content, degree of compaction, the cohesive or granular nature of the soil, and the time/rate of failure. Very little field test data are available, especially for cohesive soils. Controversy exists relative to calculating, analyzing, and applying this data to pullout design. Laboratory tests suffer from apparatus boundary effects and the influence of reinforcement extensibility on the stress distribution along the geotextile reinforcement, resulting in very conservative values (Bonapart, Holtz, and Giroud 1987). As a result, very little is known about this failure mechanism. In an attempt to obtain reliable design values, the New Orleans District conducted some field pullout tests during 1988 and

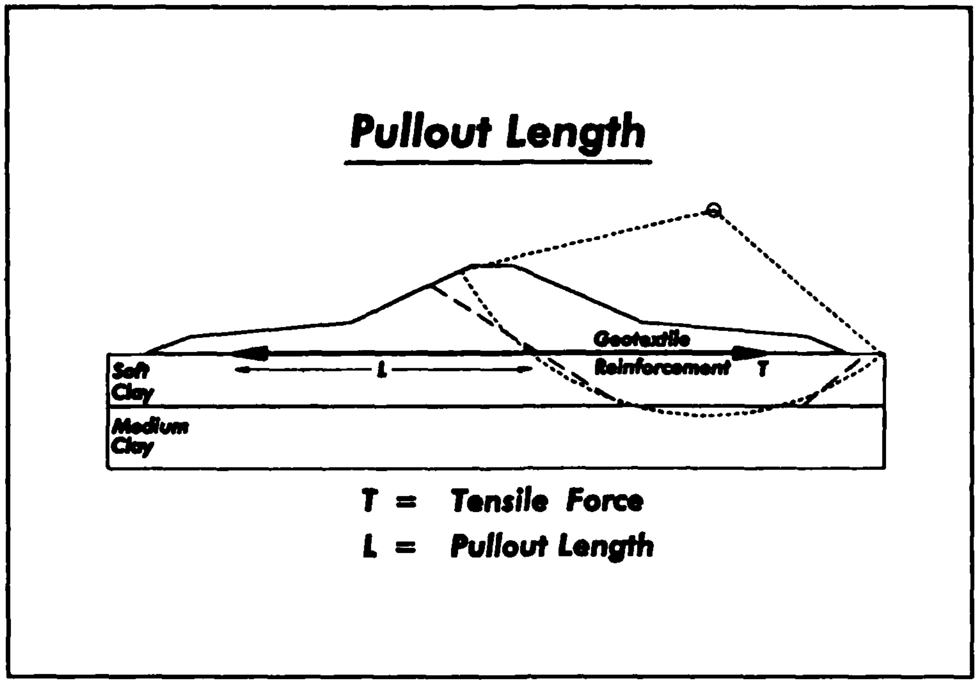

Figure 8. Pullout length

1989 in conjunction with laboratory testing at the U.S. Army Corps of Engineer Waterways Experiment Station (WES) in Vicksburg, MS. The data were analyzed by the Geotechnical Branch of the Corps of Engineer Lower Mississippi Valley Division (LMVD),the New Orleans District, and WES. Figure 9 depicts a typical design curve utilized by NOD for calculating the embedment length L. The curve relates "normal load" above the geotextile with resistance τ. An appropriate factor of safety should be applied.

The NOD field tests were conducted with a high strength woven polyester geotextile having an estimated ultimate strength of 2,900 lb/in., and weighing approximately 60 oz per square yard. The width was 6.75 ft; the embedded length was 24 ft. The fabric was instrumented with small diameter wire to measure the movement at various locations along the fabric length for each increment of load. The subgrade was essentially clay, as was the backfill material (one test was run on a sand base). The water content of the clay backfill was varied, as was the height of the embankment. The compactive effort remained constant (four passes of a D6H over 12-in. lifts; the first lift being 18 in.).

The basic field setup is schematically shown in Figure 10. The results of a typical test are indicated in Figure 11. These particular results were for a backfill height of 4.5 ft, and a water content (Wo) of 34. They show the load in tons versus the corresponding strain for each instrumented segment of the fabric. These plots were developed for 3-, 4.5-, and 6-ft embankments. The 3-ft value was chosen as a minimum design cover in the field.

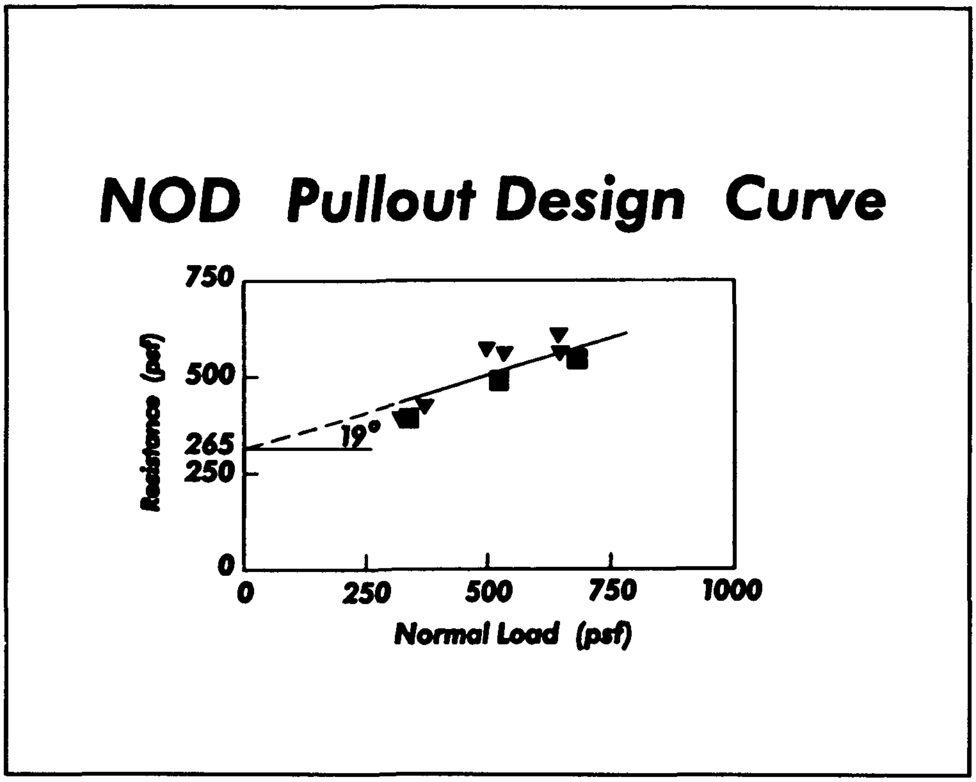

Figure 9. NOD pullout design curve

A photograph, included as Figure 12, shows the actual test site with the embedded fabric and a D6H LGP Caterpillar compacting the backfill.

The tests indicated that field pullout tests generally yielded higher pullout values as compared with existing laboratory test data. The laboratory data studied included tests by Drexel University, the Georgia Institute of Technology, the Waterways Experiment Station, and other test data published in geotechnical literature. The test results also suggest that it may be economical in the long term to run field prototype tests for projects of any magnitude. The costs to run the tests are relatively small. Even a small reduction in the length of embedment of geotextile amounts to substantial costs when applied over thousands of feet (perhaps miles) of embankments. For example, a 3-ft savings in "L" (a total of 6 ft, as an equal length of fabric must be embedded on both sides of the potential failure surface) amounts to about a quarter of a million dollars in material and placement costs for a 4.5 mile reinforced embankment (assume $16/sq yd unit cost for high strength, 1,800 lb/in. at 5 percent strain, geotextile).

Field Applications

Three different field applications were selected for discussion. The first is an example of geofabric installed on a sand base over a very soft, wet

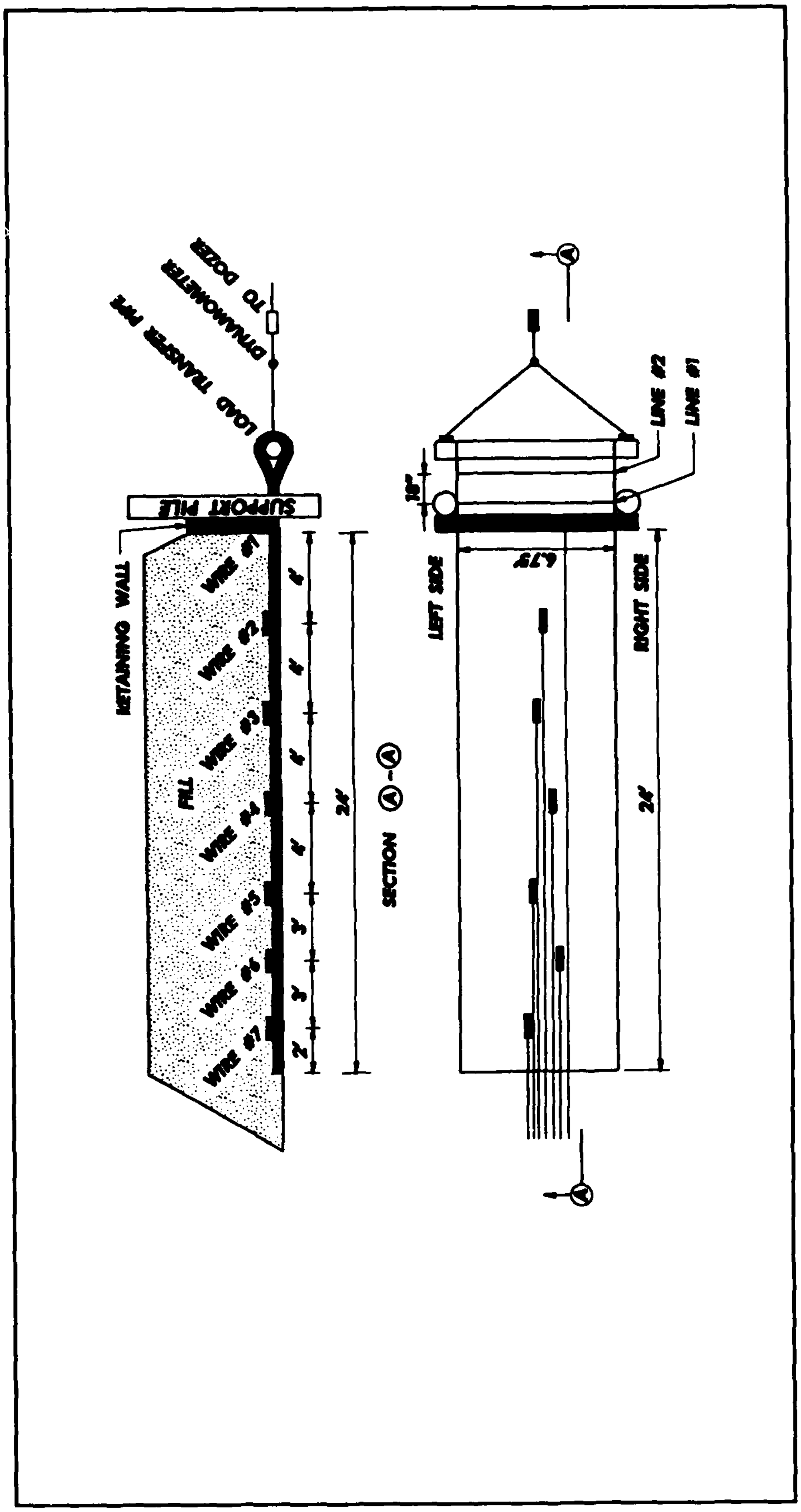

Figure 10. Bonnet Carre' pullout test

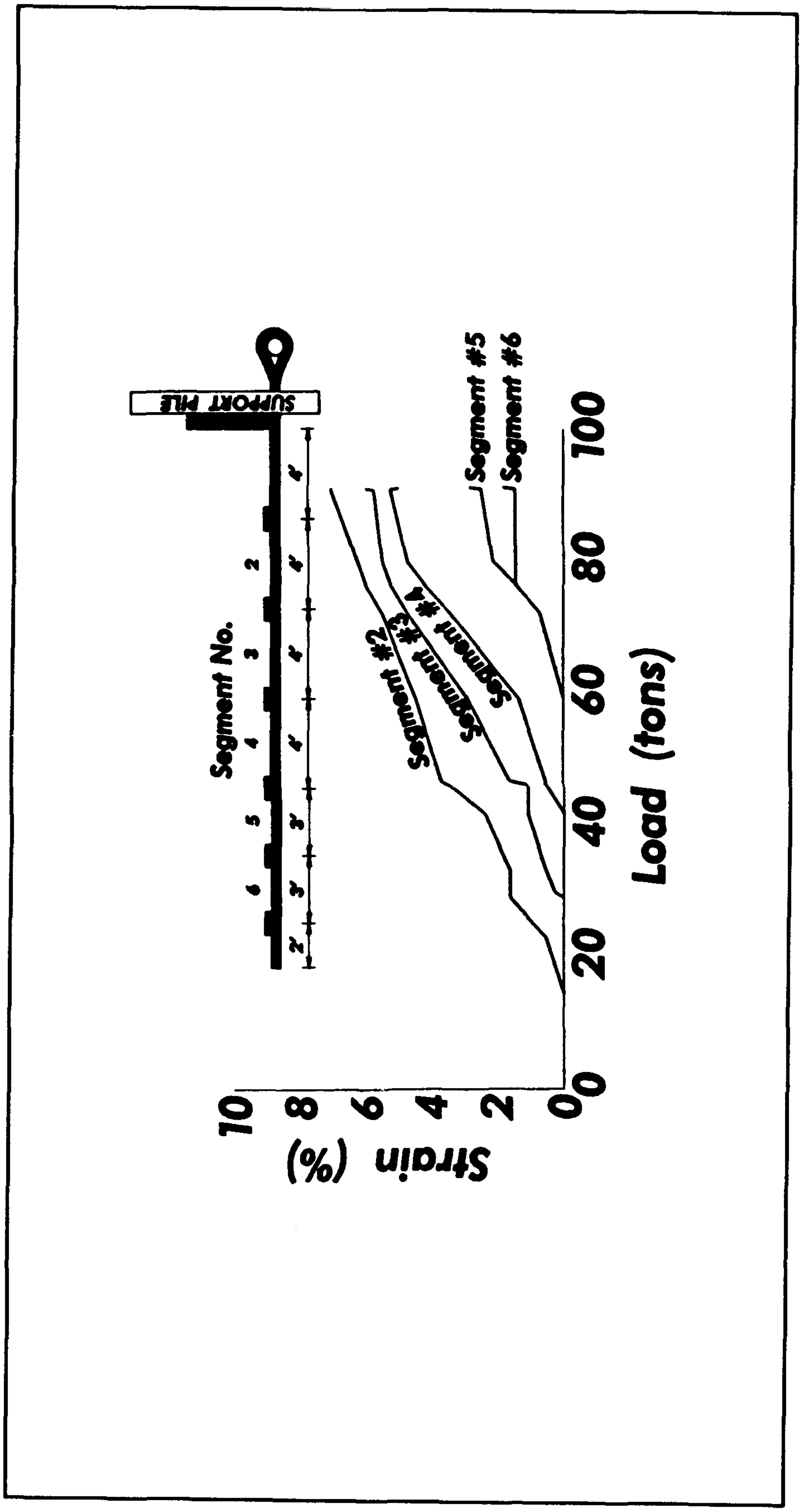

Figure 11. Bonnet Carre' pullout test, 4.5 ft, Wo = 34, 18 Oct 89

Figure 12. Bonnet Carre' test site showing embedded fabric and DGH LGP Caterpillar compacting backfill

subgrade. Field working conditions were very poor. The second case is another example of installation on a very soft, wet subgrade. However, a sand base was not used. The last instance is a case of geotextile placed on a firm dry subgrade under good field conditions. This project is currently under construction.

Sites

Airport levee. New Orleans International Airport is located to the west of the city between a large lake (Lake Pontchartrain) and the Mississippi River. To the west of the airport is a large wetland area. The east/west runway needed to be lengthened to accommodate direct international flights under all loading and weather conditions. In order to accomplish this, the runway had to be enlarged to the west into the marshlands. Since there was an existing flood protection levee between the airport and the marsh, this embankment would have to be moved further west into the wetlands to accommodate the runway extension. This would place the embankments on an extremely poor foundation and a very wet working surface.

The subsurface conditions include about 15 ft of extremely soft, organic clay with roots and wood. This material is underlain by approximately 17 to 28 ft of more very soft clay with silt layers and pockets. Beneath this layer is about 30- to 35-ft of soft clays. Medium to stiff clays (the Pleistocene Formation) are not encountered until about 75 ft below the ground surface. The groundwater table is generally at or slightly above the ground surface.

The design engineers (this was not a Federal project but required Corps of Engineer design approval) decided that the plans and specifications bid package would include two alternatives for constructing the relocated hurricane levee. The first alternative consisted of an I-wall/levee embankment. The second consisted of a geotextile-reinforced levee system constructed over a sand base. (The sand would be pumped hydraulically from the Mississippi River, and serve as a working surface for the geotextile.) All bids received were for the latter option. The stability analyses utilized for design consisted of the Army Corps of Engineers LMVD Method of Planes. (Refer to Figure 2.) The selected embankment section had a Factor of Safety of 0.93 without the fabric reinforcement, and 1.3 with the fabric. Please refer to Figure 13 for the final design section. The geotextile reinforcement had the following properties:

Tensile strength	3,100 lb/in. ultimate
	1,250 lb/in. at 5 percent strain
Seam strength	600 lb/in.

The construction sequence included the following: minor clearing and grubbing, construction of retention dikes to retain the pumped material, pumping river sand as a working base for fabric installation and levee construction, installing the fabric, and placement of clay backfill in 1 ft lifts to el 13. The levee base width was slightly less than 200 ft from toe to toe. The fabric was

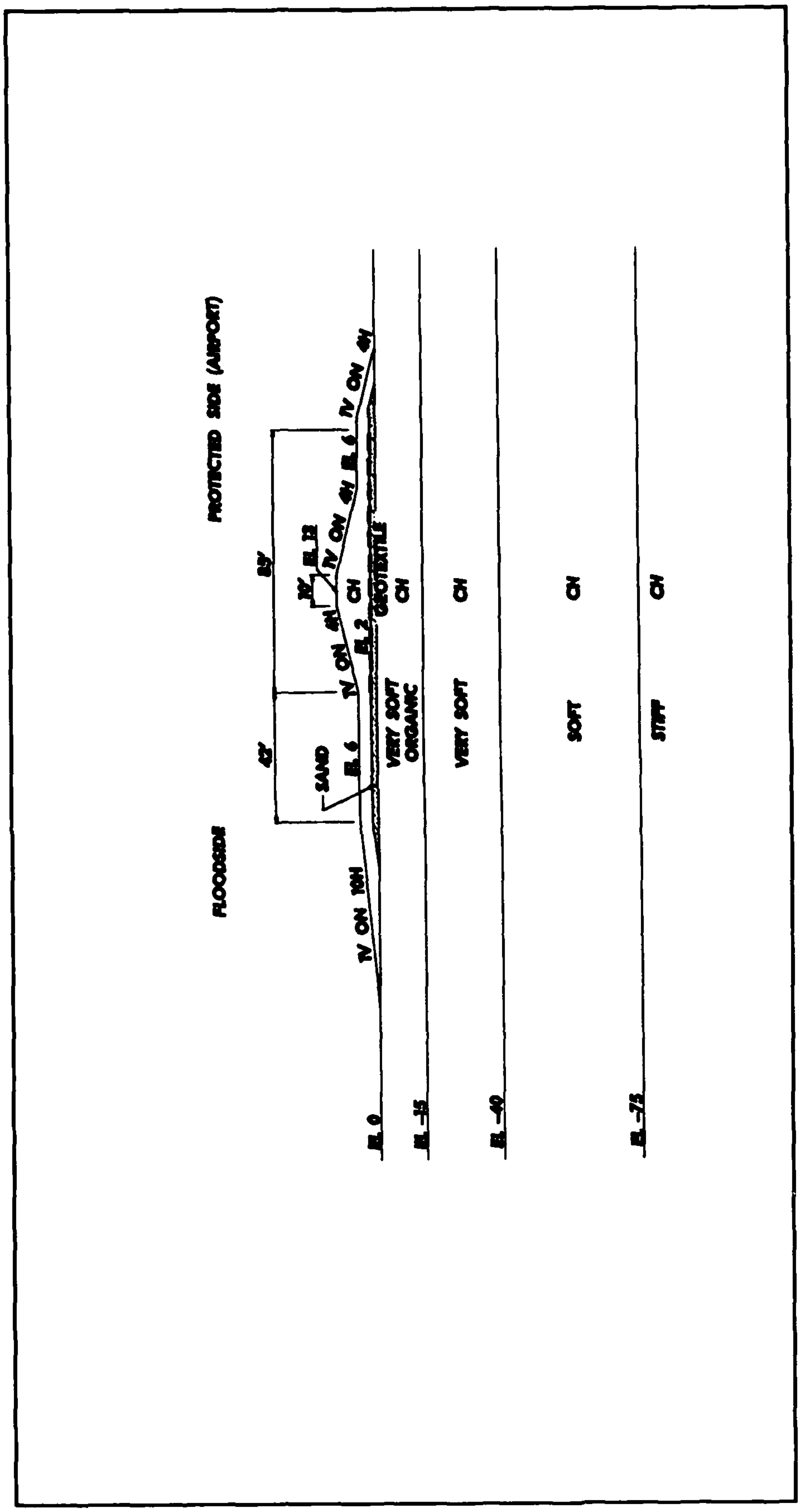

Figure 13. Airport levee section

placed in rolls, 90 ft long by 15 ft wide. Each subsequent roll was placed directly over the previously positioned fabric and unrolled; the seam was sewn; then the top layer was pulled forward into position. This sequence was then repeated.

The following observations were made upon completion of the project (relocation of the hurricane protection levee; the airport runway extension is still under construction):

a. No rotational failures were observed. This indicates that the geotextile is functioning as reinforcement, since the levee has a computed Factor of Safety of about 0.9 without it.

b. No heavy equipment was required for installation of the fabric.

c. The fabric installation did not slow levee construction. About 15 rolls of fabric were installed and field sewn on a daily basis.

d. The sand base was extremely effective, allowing the fabric to be installed with few wrinkles, and under reasonable working conditions.

Bonnet Carre' test section. The Bonnet Carre' Spillway is located to the west of the city of New Orleans. It connects the Mississippi River to the south with Lake Pontchartrain to the north. During flood stages in the Mississippi River, water may be diverted through the spillway to the lake, and thence to the Gulf of Mexico, thus protecting the city from flooding. When the spillway is utilized, the flood waters deposit huge quantities of sediment. Beneath this sediment, however, lies a relatively soft and wet subgrade that contains many cypress stumps, roots, and organic material. Note: the following technical description of the test section is, by necessity, very brief and general. A more detailed report was prepared by the New Orleans District. This study discusses the physical data collected, analyses, instrumentation data, and other observations. Further study is anticipated.

The subsurface conditions consist of an upper layer of 6- to 8-ft of silts and silty sands deposited as explained above. The underlying natural deposits, about 45 ft in thickness, are marshes consisting of clays, very soft organic clays, and peat, including roots and stumps. The bottom of these deposits defines the top of a firm material (the Pleistocene Formation). This firm material is about 50 ft below the natural ground surface. The ground water table is normally slightly below the ground surface.

Three embankment sections were designed and constructed primarily as test sections. One embankment was unreinforced; the other two were reinforced with high strength geofabric (the first with a single layer, the other with two layers separated by 3 ft of fill). All were instrumented. The embankments were designed for a Factor of Safety of 1.1, instead of the usual 1.3 against a rotational shear failure, using analyses previously discussed above. The bearing capacity Factor of Safety was also 1.1. These values were selected to facilitate failure quickly, but to have some margin of safety against failure

during construction. The test sections were constructed of uncompacted clay, with a crown approximately 16 ft above ground level (a normal height for local hurricane protection levees). The required geotextile properties for the three fabrics utilized were ("1" denotes the single layer, "2" and "3" denote double layer reinforcement).

1	Tensile strength	1,250 lb/in. at 5 percent strain 2,500 lb/in. ultimate
	Seam strength	300 lb/in.
2	Tensile strength	900 lb/in. at 5 percent strain 1,800 lb/in. ultimate
	Seam strength	300 lb/in.
3	Tensile strength	400 lb/in. at 5 percent strain 800 lb/in. ultimate
	Seam strength	200 lb/in.

The instrumentation consisted of electrical foil strain gages glued to the fabric, piezometers, inclinometers, settlement plates, and survey profiles. The purpose of the instrumentation was to measure movement prior to and after failure and to facilitate the calculation of stresses in the reinforcement, backfill, and foundation.

The sequence of construction consisted of degrading about 2 ft of silt from the ground surface to obtain a smooth base. The fabric was delivered in rolls, with all seams presewn from the factory. The fabric was unfolded utilizing hand labor, a bulldozer, and a crane. The clay backfill was placed uncompacted. After the embankments were completed (please refer to Figure 14 for an aerial photograph of two completed sections) and the instrumentation had been analyzed, failure was induced by excavating immediately adjacent to the levee toe. The unreinforced embankment exhibited the classic circular, rotational shear failure almost immediately, with a large differential movement through the crown. This may be observed in Figure 15, a photograph of the failure. The two reinforced embankments did not perform in a similar manner. They exhibited cracks in a circular pattern across the crown and backslope, but did not exhibit any significant differential movement. It is questionable as to whether these levees actually "failed," depending on one's definition of "failure." Analyses indicate that these two levees may have experienced a bearing capacity failure, or excessive lateral movement, and not the conventional rotational shear failure. The doubly reinforced levee was actually repaired (cracks filled, design grade restored) and stressed even further. No additional signs of distress were observed.

Results of the test sections proved interesting. The electrical strain gages essentially did not work. Apparently, the large strains severed the wires from the gages. Little useful data were obtained from them. The inclinometers proved most valuable, along with the other instrumentation. The embankments did not fail as predicted. The unreinforced section did fail, but after withstanding a greater stress (deeper excavation) than calculated. The two

Figure 14. Aerial photograph showing two completed sections at Bonnet Carre'

Figure 15. Photograph of failure of unreinforced embankment at Bonnet Carre'

reinforced embankments did not "fail" in a catastrophic manner, but exhibited extensive cracking and some settlement. The geofabric within apparently experienced stresses equal to about one-half of those computed at the "predicted failure." The fabric beneath the singly reinforced levee was excavated and examined and exhibited no physical signs of failure or distress.

The following are observations resulting from the test sections. It appears that the design methods and/or assumptions employed were perhaps conservative. Bearing capacity is a factor that must be carefully considered. The "failure" of reinforced levees may be less severe than unreinforced embankments, and perhaps repairable. The stress-strain characteristics of the soil and fabric system must be given close attention. The use of geotextiles to stabilize embankments on soft foundations does appear to be an economical and viable design alternative.

Jefferson lakefront levee. The Jefferson Parish Lakefront Hurricane Protection Levee is located on the south bank of Lake Pontchartrain, and is about 10.5 miles in length. It is designed to protect Jefferson Parish, a large and densely populated suburb located to the west of New Orleans, from flood waters of Lake Pontchartrain during a hurricane that generates strong southerly winds. The levees had been in place approximately 35 years or more when it was determined that the design grade had to be raised about 6 ft. A detailed engineering study concluded that it was more economical to raise the existing levee in place. Geofabric was utilized to limit the width of the levee because of dense residential development immediately adjacent to the levee. Thus, a dry and firm foundation was available.

Over 100 borings were taken to define the subsurface conditions. About half of these borings were "undisturbed" types. The subsurface consists of an upper layer of clay fill material placed as a levee over 35 years ago. Underlying this levee fill is approximately 50 ft of very soft to soft clay containing varying amounts of peat and organic matter. Underlying this stratum is the Pleistocene Formation, consisting of stiff clays. The top of this formation is about 50 ft below sea level.

The final design section, illustrated schematically in Figure 16, consists of an earthen compacted clay embankment reinforced with geotextile to an elevation of about 16 ft above sea level. The levee was designed for about a 1.3 Factor of Safety against rotational failure. Stability berms were generally required and averaged about 30- to 50-ft in length. The strength of the fabric was used to limit the size of the berms, as there were severe right of way limitations on the protected side toe. The pullout embedment for "T" was determined utilizing the data developed in conjunction with the pullout tests discussed above. The design strength of the geotextile varies throughout the project length. However, the figure shows a typical value of about 1,250 lb/in. at 5 percent strain. This value was also limited to about 50 percent of ultimate strength to minimize creep. The required seam strength was 300 lb/in. (a low value) because of the firm, dry working surface.

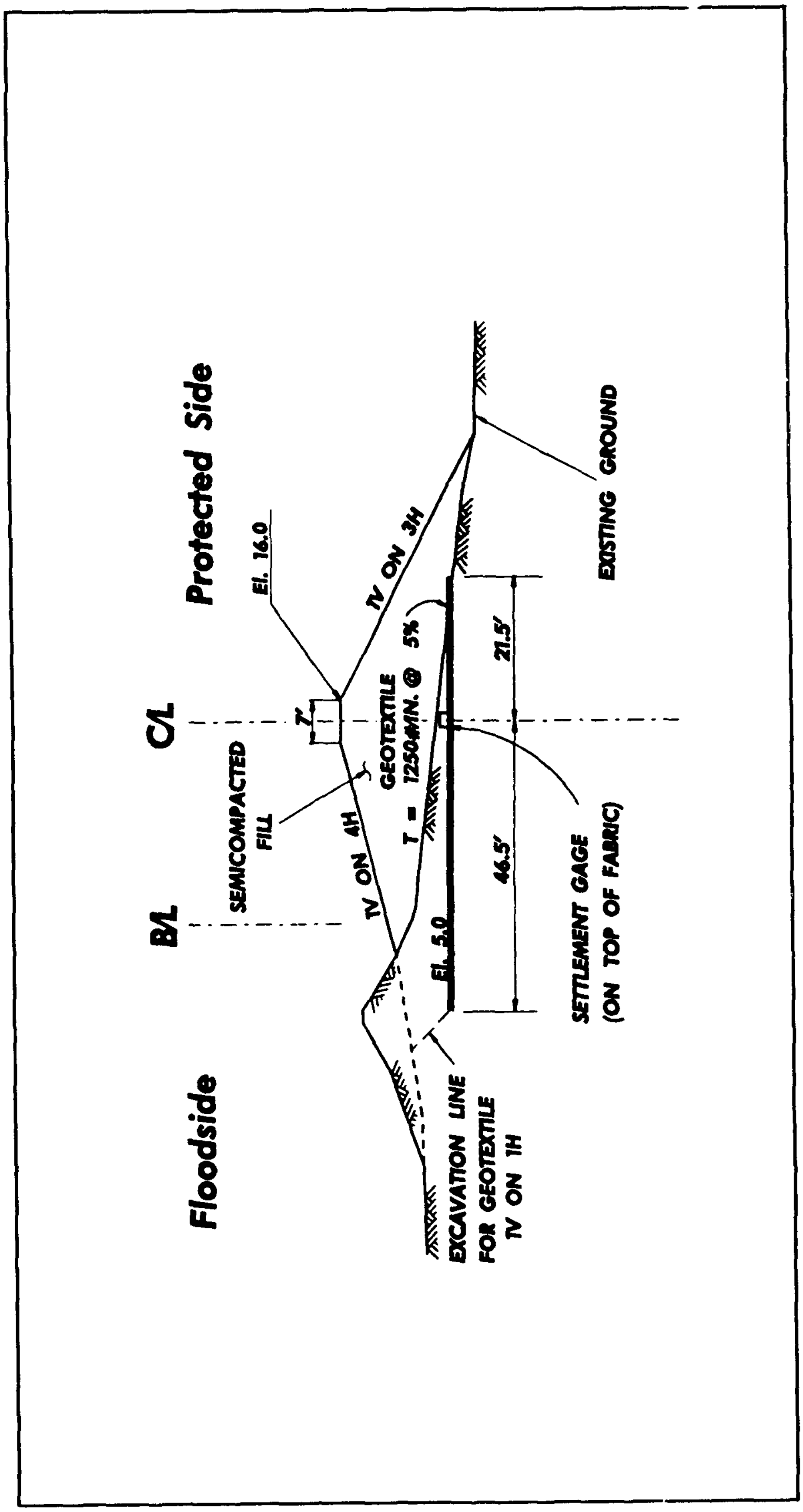

Figure 16. Design section, Jefferson lakefront

The sequence of construction was as follows. The existing embankment was degraded to a smooth surface at about el. 5. The degraded material was placed adjacent to the excavation on the protected side. The fabric was placed, and then backfilled with the stockpiled material. Additional material was hauled from a remote site (Bonnet Carre' Spillway) by truck. The backfill was placed in 1 ft lifts and compacted. The total project was divided into five reaches because of varying soil conditions and ground profiles. Each reach is to be constructed under a separate contract. Currently, the project is still under construction.

The following observations are pertinent. After construction of several reaches, a rotational failure was noted in the one unreinforced reach. This was probably due to an over estimation of the in situ foundation strengths. No rotational failures were observed in the reinforced embankments constructed to date. No heavy equipment was required for the installation of the fabric. The fabric was easily and conveniently placed on the firm, dry working surface (degraded existing embankment).

Conclusions

The relatively recent introduction of high strength geotextiles as reinforcement for embankments on soft foundations has opened up a new and challenging design alternative for the geotechnical engineer. Many theories and design procedures have been advanced as the recent result of research and actual field experience. However, a great amount of work is still needed to adequately define many unknown quantities, as total agreement on the current "state of the art" is not yet in sight.

From a design perspective, it is readily apparent that such factors as soil properties (shear strength, compactive effort, water content, etc.), fabric properties (tensile strength, modulus of elasticity, creep resistance, aging properties, and others), and types of analyses employed are vital to the ultimate performance of the reinforced embankment.

From a construction perspective, recent field experience indicates that reinforced embankments on soft foundations are a practical and economical solution to difficult problems. The successful completion and performance of many projects also indicate that current design methods and assumptions are "within the ballpark."

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Use of Soil Nailing for Slope Repair

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New Orleans District

Introduction

This paper presents the results of a geotechnical investigation to determine causes and potential remedies for a 700-ft section of the Mississippi River bank in the Baton Rouge, Louisiana, area which has been experiencing a creep failure into the river.

Historical Perspective

The bank at this site has developed over the last century through a series of natural and man-made loadings (refer to Figure 1). It appears that 100 years ago the bank was very steep, dropping off at the approximate present location of Front Street and dipping to the top of the Pleistocene clay. Sometime in the late 19th century the railroad was built at the top of the bank. The bank scallop in this area was filled in with rubble--a combination of gravel, soil, coal, and brick. After this period, the river accreted silt over the rubble to form the flat bank which exist today. Over the years, riverward creep, and vertical and horizontal consolidation of the rubble pile and the foundation soils have contributed to the displacement of the railroad tracks. In 1934, the Corps raised the level of flood protection and relocated the tracks on top of the levee. Continued settlement of the tracks has brought about the raising of railroad ballast periodically throughout the years. No records of how much or how often the tracks have been raised are available. Then, around 1950, the area landward of the levee/railroad embankment was filled in and Front Street was constructed, completing the present configuration of the area. Front Street has required repairs on about a yearly basis in this area due to cracking and deformation settlement. Compounding the problems associated with the random fill, the accretionary deposits impede drainage in the bank, which, at this site, is the natural avenue of storm drainage for a substantial area of landward watershed.

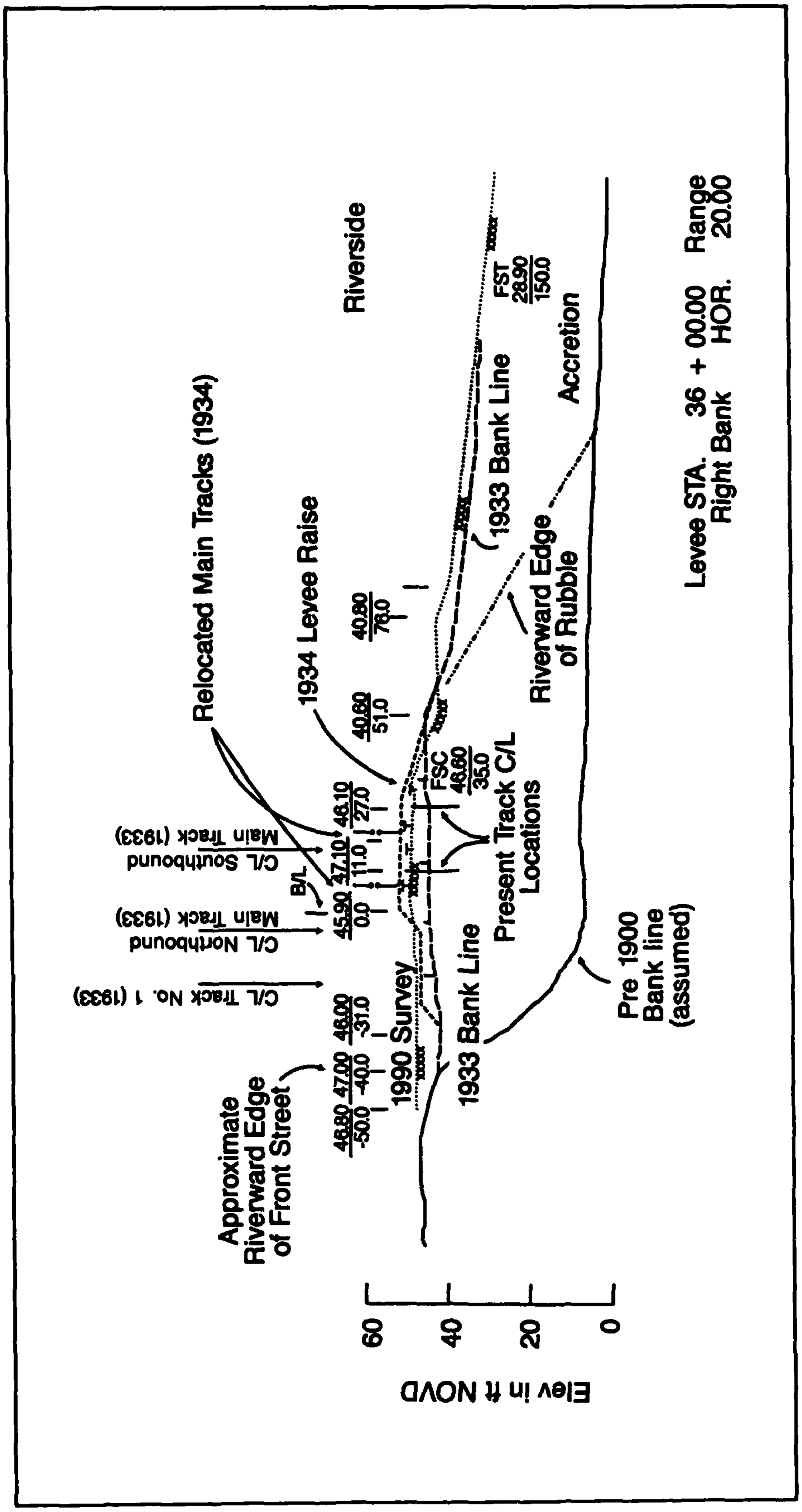


Figure 1. Site development

Background

The flood protection levee in this area is presently about 3 ft deficient in freeboard grade, and continues to settle and creep riverward. Levee design grade for this area is elevation +50 NGVD, and the low water reference plane is elevation +2.5 NGVD. The slope of the riverbank at this site is very flat, about 1V on 9H, and the possibility of a catastrophic failure is considered to be remote. The bank is not revetted, however the low water line has not changed significantly in the last 40 years. In 1987 the New Orleans District initiated a 3-year study plan to determine how to best re-establish design grade freeboard for this area.

Geotechnical Investigations

Thirteen borings were made, along with a Pulsar (sonar) subsurface contour mapping (see results at Figures 2 and 3), to better define the extent and properties of the deposit. Ten slope inclinometers and six piezometers were installed and monitored over a 3-year period. A well was installed into the deposit and a pump test was performed to determine the feasibility of a drainage solution to the problem. In addition, comparative cross-section surveys were taken. See Figure 4 for a section view location of the instrumentation.

The boring and Pulsar mapping indicated that the deposit extends approximately 700 ft along the alinement of the flood protection and is approximately 50 ft thick at its deepest point. Triaxial shear tests were performed on representative samples from the borings to determine shear strength parameters for design.

The slope inclinometers were placed in six locations. Two of the instrumented locations experienced movement of sufficient magnitude that replacement was necessary. The inclinometers near the center of the deposit recorded a total of 9 in. of riverward movement over a 3-year period. The inclinometers that were installed near what are assumed to be the upper and lower limits of bank movement recorded only 1.1 in. and 0.8 in. of movement in slightly over 1 year. The movement of the bank appears to be primarily perpendicular to the baseline although a slight upstream component of movement was noted. The depth of the slip surface was well defined by the inclinometers. These readings formed the basis for the selection of the critical arc for use in stability analysis. Plots of a typical inclinometer readings are shown in Figure 5. Figure 6 shows the location, magnitude, and depth of movement for all the inclinometers. The comparative cross-section surveys did not indicate any underwater bank movement.

Piezometric heads were recorded in six locations and at varying tip elevations. The most valuable data are the plots of piezometric head in the bank with respect to river stage fluctuation (see Figure 7). An obvious drawdown

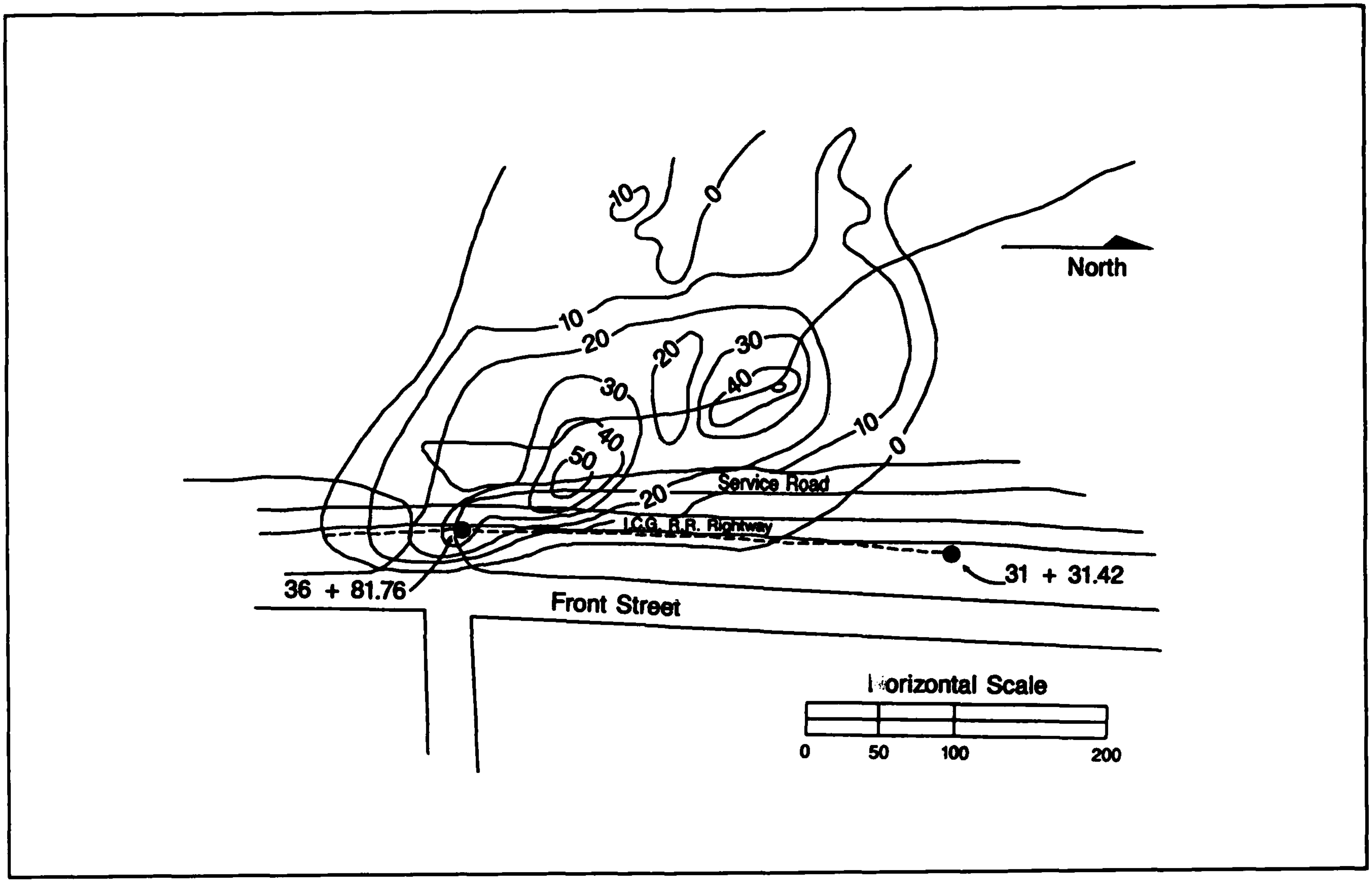


Figure 2. Gravel isopach contour, thickness equals 10 ft

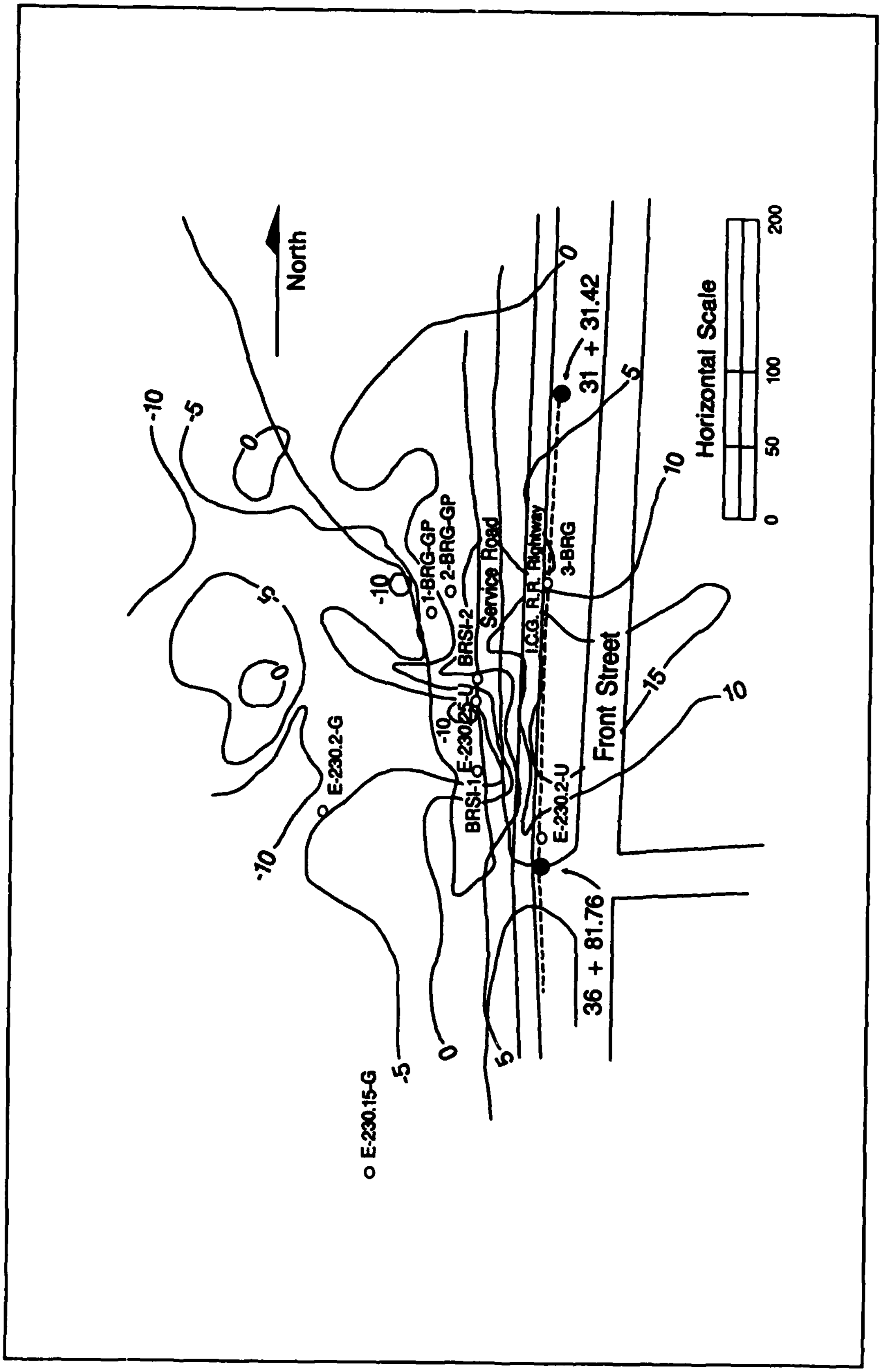


Figure 3. Pleistocene contour, interval equals 5 ft

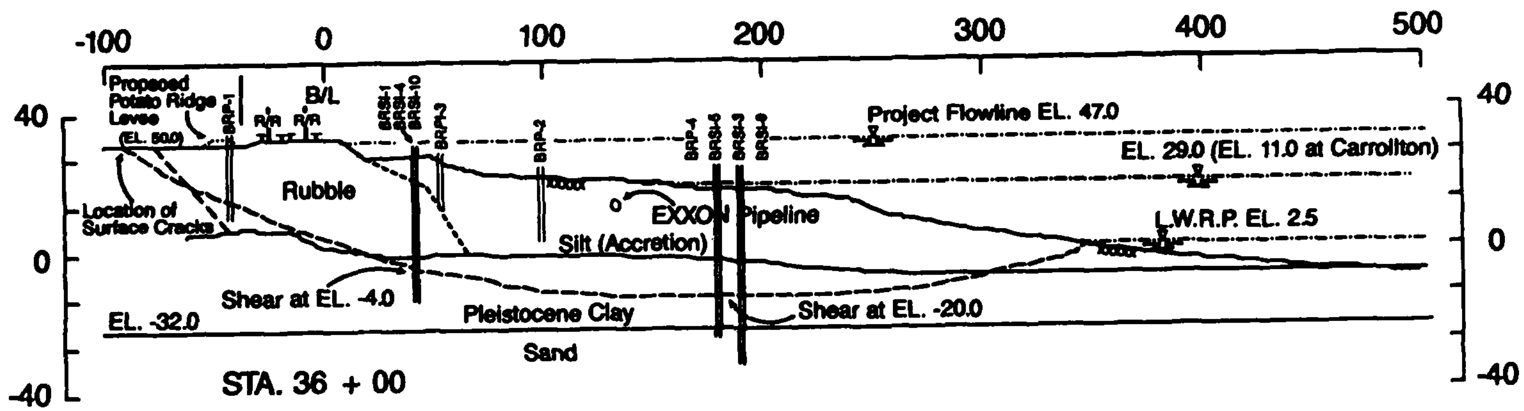


Figure 4. Instrumentation section view

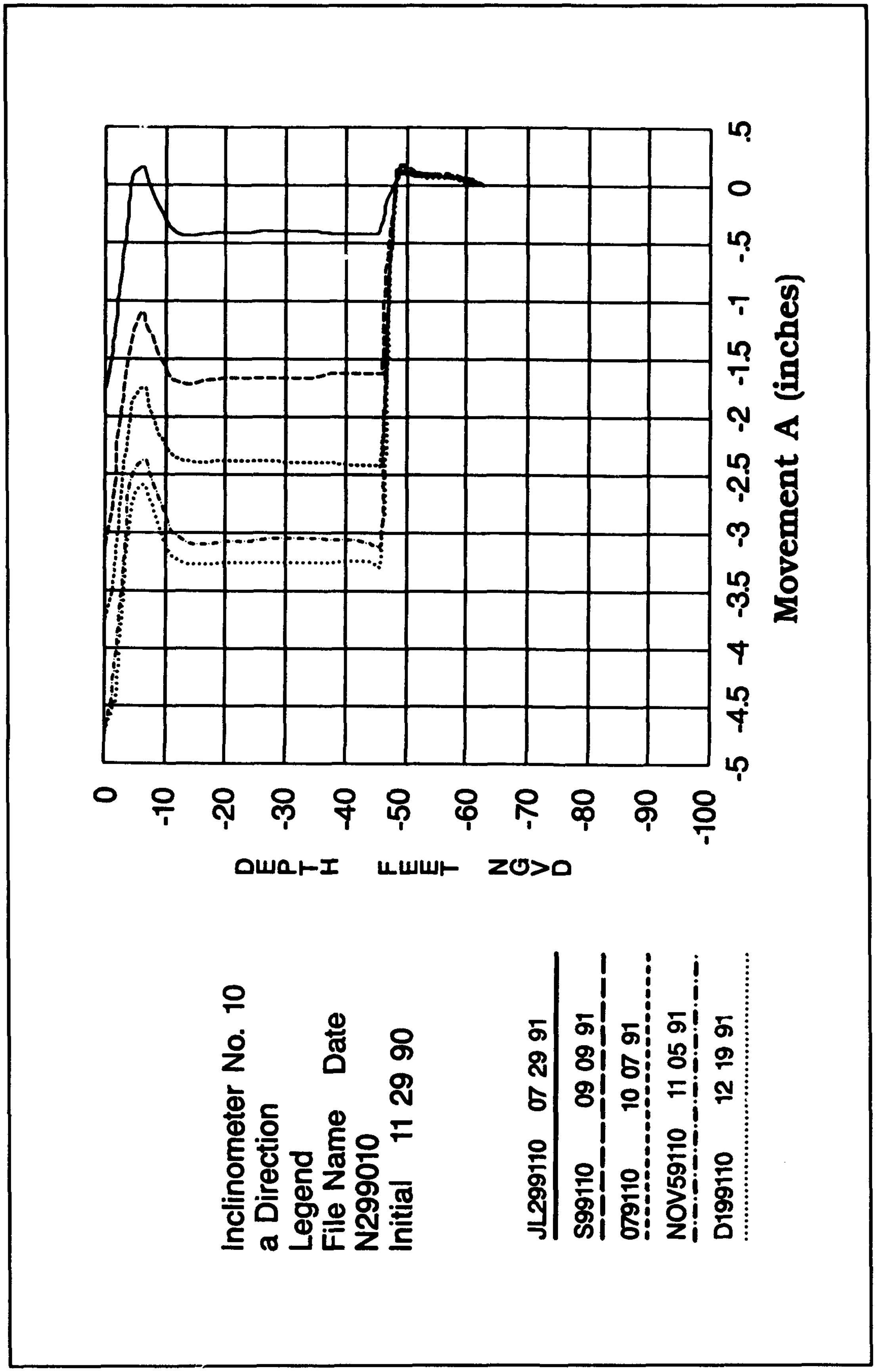


Figure 5. Inclinometer

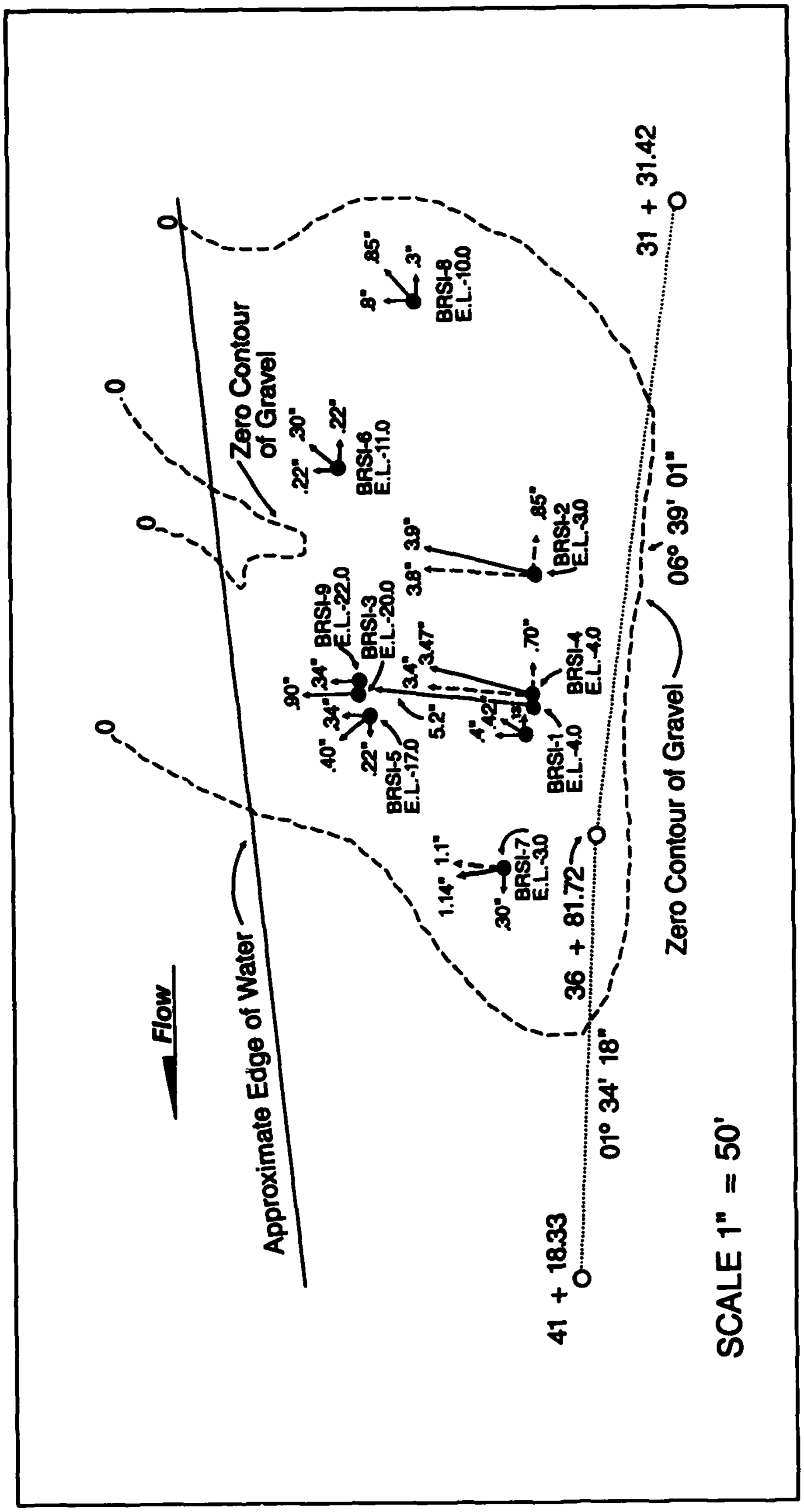


Figure 6. Inclinometer, August 1991 data

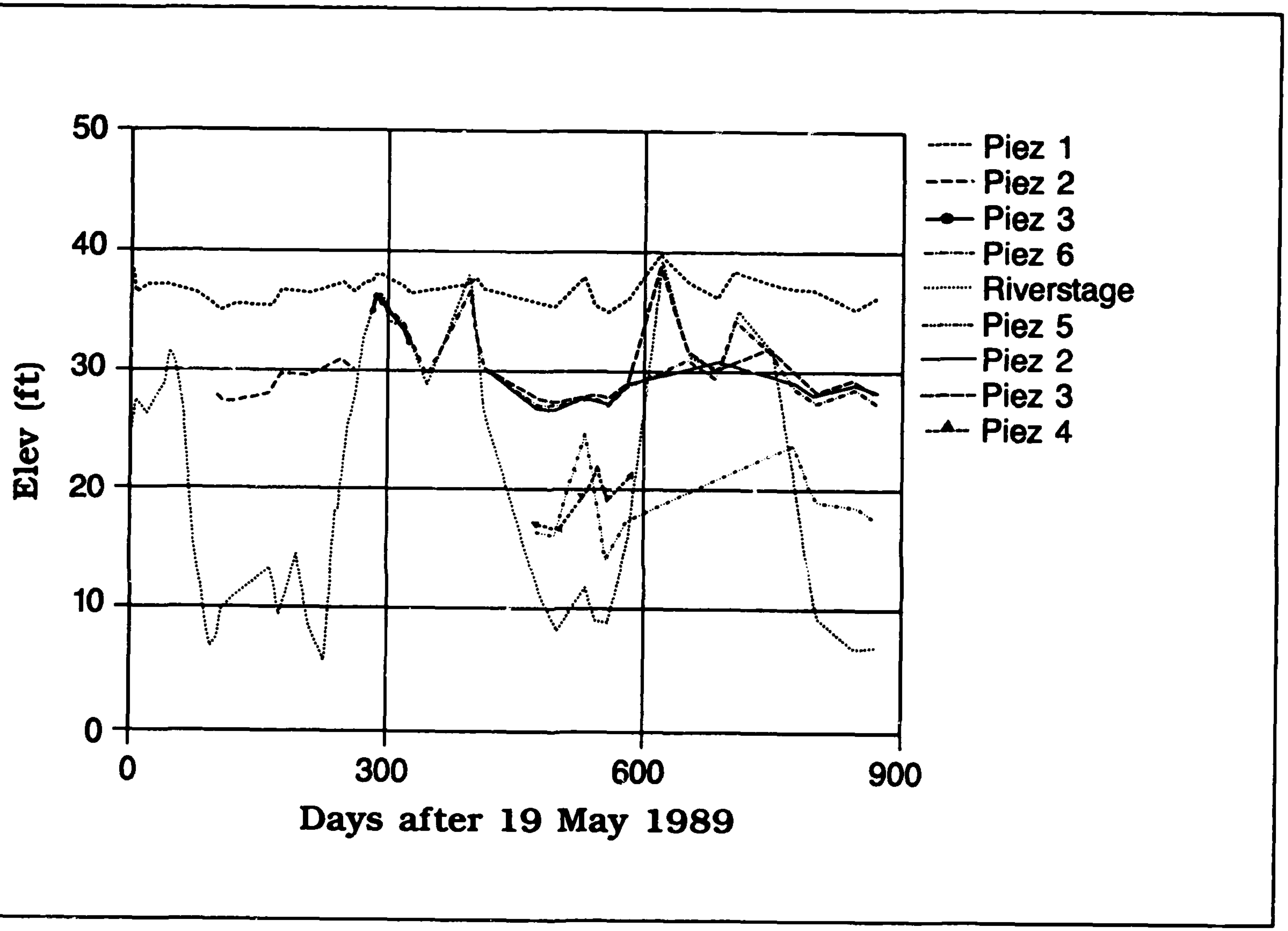


Figure 7. Baton Rouge front, piezometer readings

lag exists. The bank foundation soils, which are not free draining, hold a perched water table approximately 5- to 10-ft below the bank line. The excess head appears to remain high with time, even in periods of extended low river. Comparisons were also made of piezometric head with respect to rainfall. Correlation of the data was inconclusive although visual inspection of the site verifies that the site acts as a natural drainage avenue for the area just landward of the embankment. No geologic evidence exists which would imply a hydraulic connection between this area and nearby Capitol Lake and there is no indication of artesian pressure in the area. There is a storm drainage system which runs parallel to Front Street between the street and the embankment. Inspections of the system were performed by the City of Baton Rouge. No major leaks were detected.

A pump test was performed on the bank to determine if the water level in the upper bank could be pulled down by pumps or drains. The water level in the well, which was founded in the rubble deposit, was pulled down 14 ft in a short period of time. Only 1 gal/min pumping rate was required to sustain the drawdown. However, there was no response in any of the piezometers. This indicates that either the water is being recharged at the same rate, or, more likely, the permeability of the rubble deposit is not great enough and consistent enough to permit drainage. After completion of the pump test, the alternative plan to provide drainage of the area as a part of remediation plan was abandoned.

Stability Analysis

Initial attempts to model the bank creep were based on available unconsolidated-undrained (Q) triaxial compression tests. A plot of these test data and the selected design line for the clays are shown in Figure 8. Stability analyses indicated that, even for the most extreme loading, sudden drawdown with 100 percent lag from flowline to the low water reference plane, levee and bank safety factors were well above acceptable limits for the documented failure plane.

The slope inclinometer data show that the slip surface dips into the Pleistocene clay deposit. Additional shear testing was performed on the Pleistocene clay to determine the long term (S) shear strength and the lower boundary of the shear strength based on repeated shear tests. Values of ϕ = 19-deg peak and ϕ = 14-deg residual were obtained. Repeated shear produced an ultimate value of ϕ = 7.5 deg for one sample.

The characteristic pattern of bank movement over the 3-year monitoring period was expected--that movement accelerates during dropping river stages and slows as the river rises. In keeping with this pattern, a series of stability analyses were performed on the existing bank, using a phreatic surface in the bank corresponding to the piezometric data recorded for the river stage (el 15.0) above which bank movement slowed. Using parameters for the rubble consistent with those for gravel (γ = 130 pcf, ϕ = 40 deg), a stability

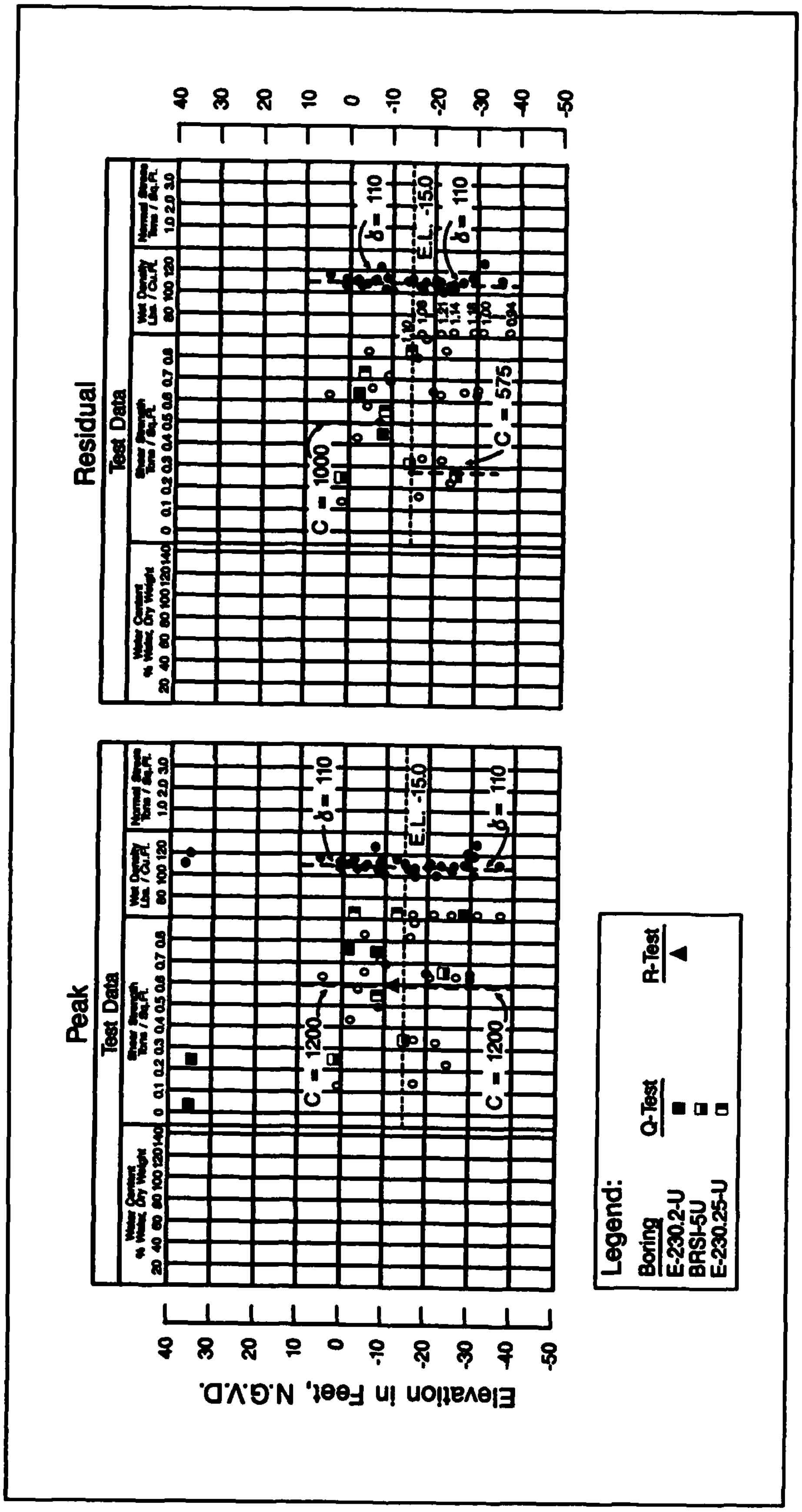


Figure 8. Shear strength and wet density

analysis was performed for an arc passing through the three known points of failure, as determined from the slope inclinometer data and observed ground surface cracks landward of the flood protection. Even using residual shear strengths as low as $\phi = 2$ deg, $c = 0$ for the Pleistocene, the analyses resulted in safety factors above 1.1, which does not model the failure.

In order to correctly model the failure (S.F. = 1.0), a back-figured shear strength of $\phi = 11$ deg, $c = 0$ was used for both the clay and the rubble. Although the assumed failure circle passes through the rubble, the Pleistocene/rubble interface is so close to the circle that it is possible that the shear surface may deviate from the circular pattern to pass through the Pleistocene clay. This analysis is shown in Figure 9. All bank remediation alternatives were based on these properties for the assumed failure surface.

Remediation Alternatives

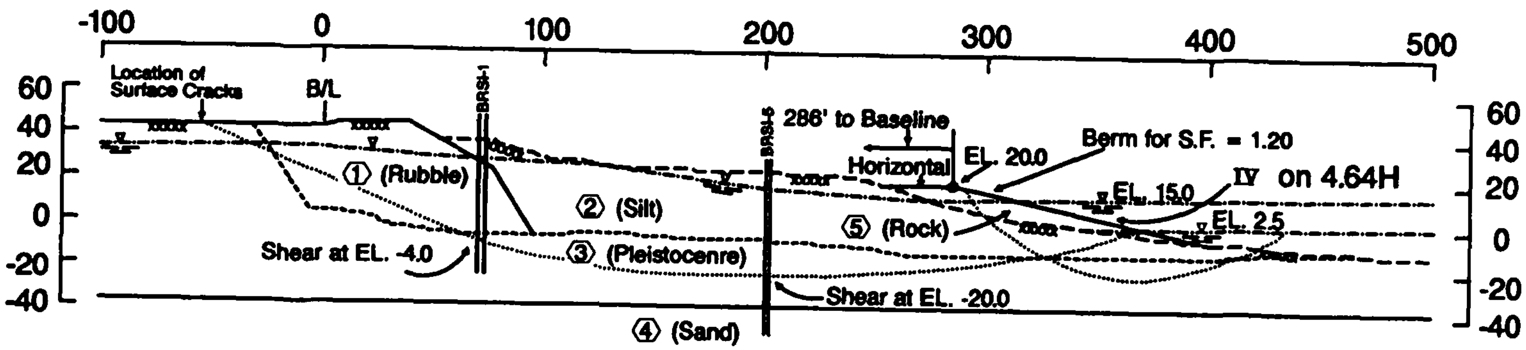
Because lower boundary shear strength values were used in the design of bank repair measures, a design safety factor of 1.2 is considered to be adequate. Another consideration in the selection of this safety factor is that the bank movement is a long-term creep condition which does not immediately jeopardize the flood protection.

Stability berms are the most conventional repair measures for this type of problem. Cost and hazard to navigation are two factors weighing against use of an underwater rock berm here. Shown in Figure 9 is the design for a rock berm. A preliminary cost estimate for construction of a 700-ft wide berm is $518,000.

Soil nailing is also considered to be a viable alternative. Using the same parameters as were used for the rock berm design, a soil nail system was designed to resist the unbalanced driving forces and restore a safety factor of 1.2. Several options are available regarding type, size, and spacing of the soil nails. The lengths and depths are functions of the type, size, and spacing chosen. The soil nails would be driven in a corridor between 80 ft and 100 ft riverward of the baseline. They would be placed at a depth such that the center of the nails would be at the failure surface, el -4 to -10. A preliminary cost estimate for stabilizing this bank with W12 X 120 soil nails, 24 ft long, placed in 2 rows at 12-ft spacing (requires a total of 117 nails) is $145,000.

Soil Nail Design Methodology

The methodology employed is described as the National Cooperative Highway Research Program Report 290, entitled "Reinforcement of Earth Slopes and Embankments." This methodology is based on a design approach proposed by Winter et al. (1983) for creeping cohesive soils. The basic assumptions are: the mobilized shear stress (τ) in the slope equals the shear strength



Design Case

Condition	Soil Properties ① γ	① ϕ	① C	② γ	② ϕ	② C	③ γ	③ ϕ	③ C	Circle* Data X	Y	R	River* Stage (Feet)	Factor of Safety	Resisting Force lbs./ft.
1	130	11	0	117	28	0	110	11	0	212	524	544	15.0	1.02	
2	130	11	0	117	28	0	110	11	0	212	524	544	2.5	0.93	
3	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.09	
4	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.30	
5	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.20	
6	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.63	
7	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.00	15,000
8	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.20	43,000
9	130	11	0	117	28	0	110	11	0	212	524	544	2.5	1.30	55,000

⑤ (Rock) $\gamma = 140$, $\phi = 40°$, C = 0
*Reference to N.G.V.D., or Baseline Datum
R = Radius
γ = P.C.F.
C = P.S.F.
1 = Estimated Elevation at which bank stops moving Based on Inclinometer Data
2 = Low River Stage
3 = Dewatering bank to E.L. 20.0
4 = Rock Berm - Failure along Shear Surface
5 = Rock Berm - Failure along Shear Surface
6 = Local stability for Rock Berm with F.S. = 1.3 along Shear Surface
7,8,9 = Resisting force for desired F.S.

Figure 9. Stability analyses

(S_u) associated with a particular initial strain rate (E_i); each nail has an area of influence (Seq); and nails reduce the strain rate from (E_i) to (E).

From these assumptions it follows that if the inclusion of soil nails reduce the strain rate, then the mobilized shear resistance (τ) in the slope must also decrease by an amount equal to the resisting shear forces developed in the nails. This reduction of mobilized shear resistance ($\Delta\tau$) is:

$$\Delta\tau = -S_u(E_i)\, I_v \log_e \left[\frac{E}{E_i}\right]$$

where

$S_u(E_i)$ = Undrained shear strength at E_i
I_v = Viscosity Index determined from undrained triaxial shear tests after Leinenkeegal [1976]

For this reduction ($\Delta\tau$), the nail must support a force equal to τ time Seq. This force is balanced by a resisting force (Q) provided by the nail. Hence:

$$Q = Seq(\Delta\tau)$$

These equations imply that for many cohesive soils, the rate of creep can be reduced by an order of magnitude if 10 percent of the driving forces is supported by nails.

Figures 10 and 11 will facilitate description of the design procedure. The goal is to optimize the spacing between nails in a row perpendicular to the slope (a) and also the length of nail (h) extending below the creep and noncreep interface. This is done by assuming the upslope and downslope spacing between nails (L) and also the nail diameter (d). The length of nail (h) must also satisfy the following equation in order for the entire length (h) to be effective in transferring load from the creeping to the noncreeping zones:

$$h \leq 3\sqrt[4]{4EI/K_s d}$$

where

EI = Bending stiffness of the nail
K_s = Subgrade modulus of the soil

The dimensions x_o and y_o are defined as shown in the simplified pressure diagram, Figure 11, such that $x_o + y_o = h$. Also, the magnitude of force (P) exerted on the nail per unit length is constant with depth. By taking moments

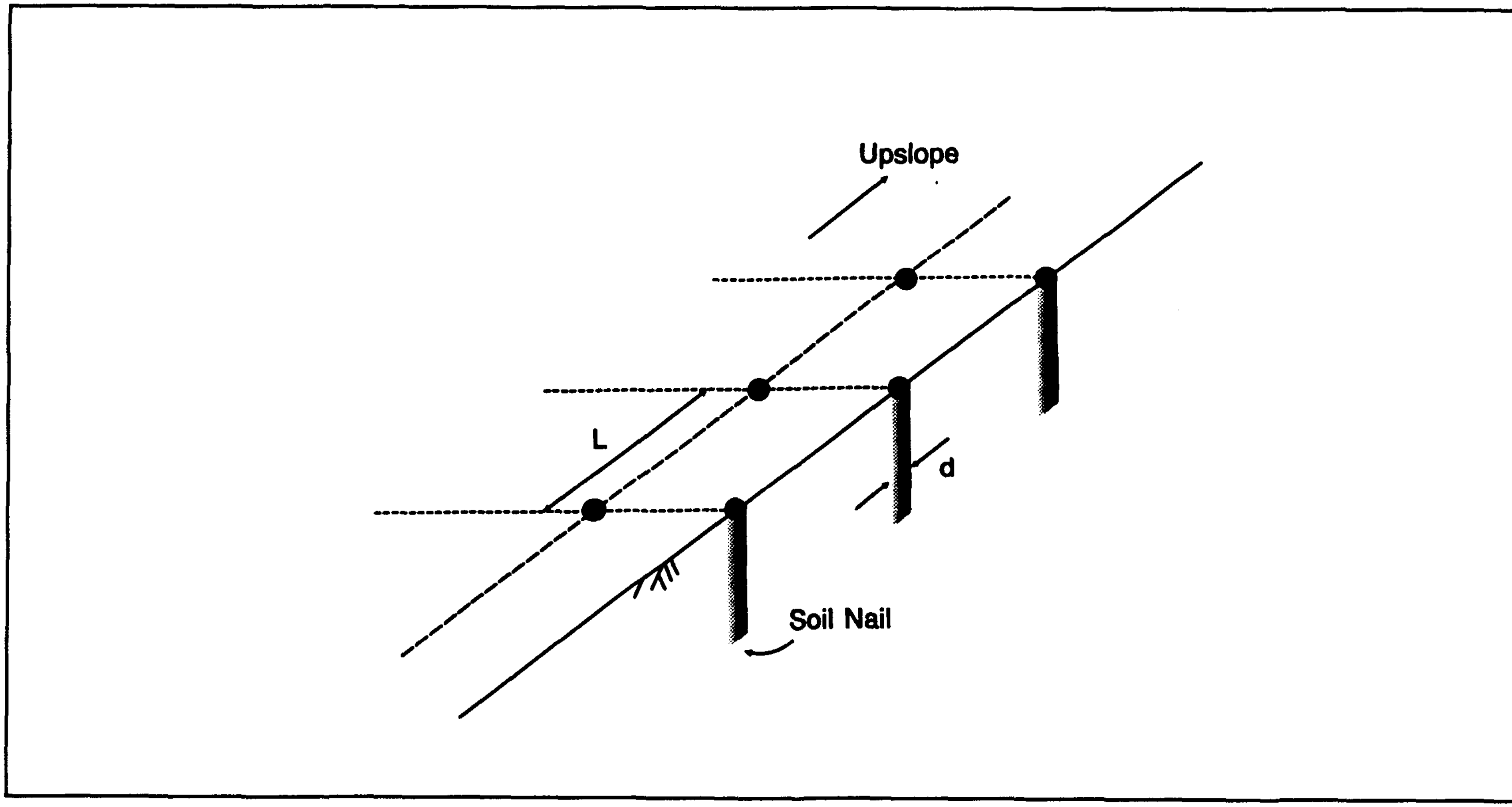


Figure 10. Soil nail design parameters

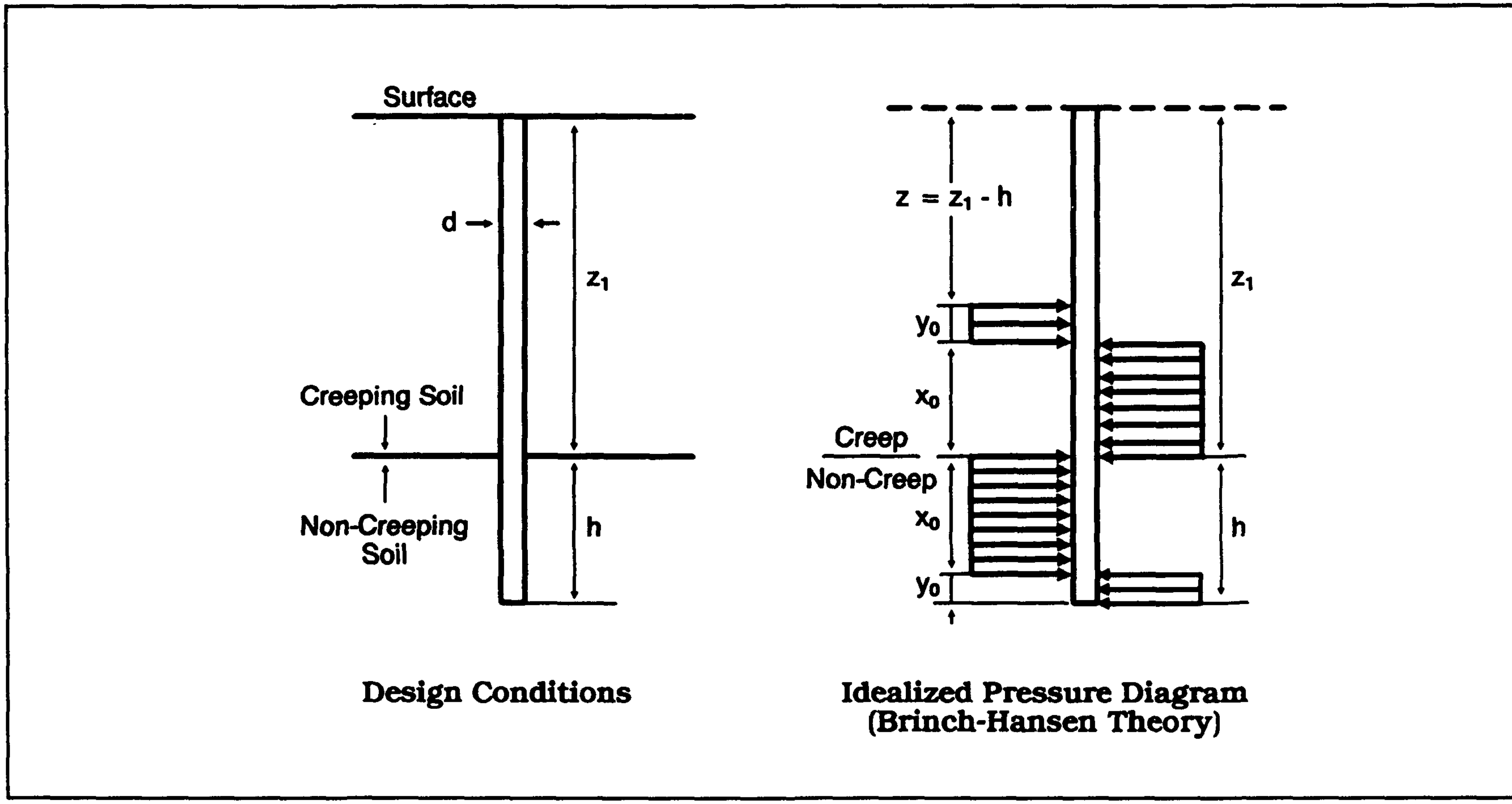


Figure 11. Soil nail design parameters, Brinch-Hansen theory

about the interface of the creeping and noncreeping zones and equating to zero, the following equation can be written:

$$Py_o \left[\frac{y_o}{2} + x_o\right] = P\left[\frac{x_o^2}{2}\right]$$

With $x_o + y_o = h$, x_o and y_o can be shown to equal 0.707h and 0.293h, respectively. Thus, the total shear force (Qs) transmitted from the creeping to the noncreeping zone by a single nail is:

$$Qs = P\,(0.707 - 0.293)\,h = 0.414\,Ph$$

By constructing a bending moment diagram, it can be shown that the maximum bending moment acting on the nail is:

$$Mmax = 0.085\,Ph^2$$

To optimize a and h, the maximum bending moment in the nail should equal the allowable bending moment (Ma). Hence:

$$0.085\,Ph^2 = Ma$$

$$\textit{Substituting } P = \frac{Q_s}{0.414h}, \textit{ gives } 0.206\,Q_sh = M_A$$

Combining with the equations from paragraph 17 above, and allowing Seq. = aL, three equations with three unknowns can be written as follows:

$$Q_s = -S_u(E_i)\,I_v\,a\,L\log_e\left[\frac{E}{E_i}\right] \qquad (1)$$

$$-\left[S_u(E_i)\,I_v\,a\,L\log_e\left[\frac{E}{E_i}\right]\right]0.206h = M_A \qquad (2)$$

$$P = \frac{-S_u(E_i)\,I_v\,a\,L\log_e\left[\frac{E}{E_i}\right]}{0.414h} \qquad (3)$$

The three unknowns are P, a, and h. By assuming a value for h, the value of P can be obtained from the design curve shown in Figure 12. With P and h known, a can be computed from Equation 2 above, while Equation 1 may be used to verify that the shear strength of the nail is not exceeded. The h, P, and a values can then be optimized with an iterative procedure. The optimal

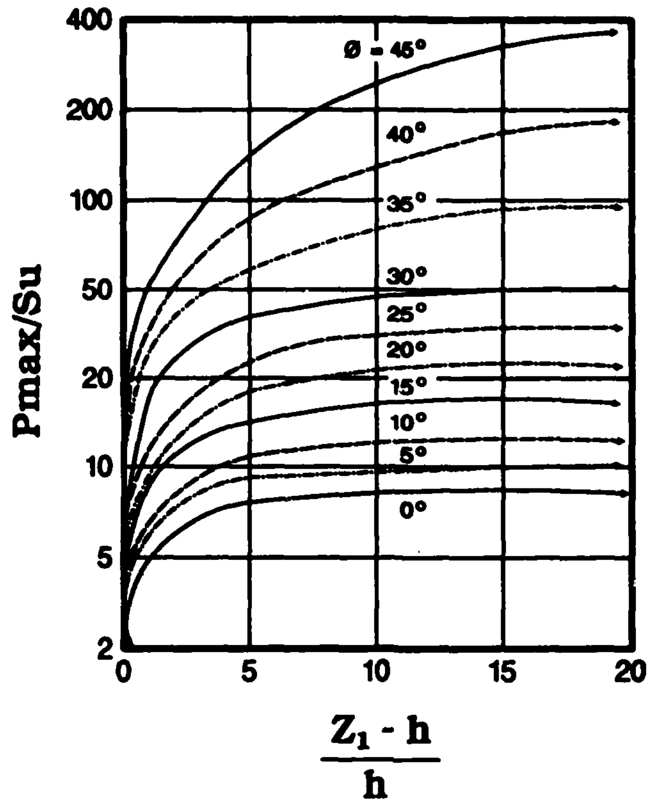


Figure 12. Brinch-Hansen design curves

situation is obtained when the selected value of h and the calculated values of P and a approximately satisfy Equation 3.

Summary and Conclusions

The sporadic but progressive creep movement is the result of long-term overstress of the bank caused by the added loads imposed by the rubble dump, the railroad, the flood protection, and Front Street. The movement may have been precipitated by the flow of water out of the bank during falling river conditions and significant rainfall events. The movement has been going on for years, and, left unchecked, it will probably continue at the rate of about 3 in. per year. The rate of bank movement represents no immediate threat to the flood protection.

Based on the data obtained, four important determinations were made:

a. The length and depth of the failure surface were accurately defined.

b. The effect of the river stage on the movement was recorded.

c. Drawdown of the perched phreatic surface through a series of pumped wells and/or drains does not appear to be a viable solution.

d. Classical stability analyses and strength parameters do not model the creep failure.

Soil nailing failing slopes is economically viable compared with conventional stability berm repair. The simple approach used for determining the soil nail system herein appears to be adequate for practical design, rendering reasonable results. This is especially true when it is considered that additional nails can always be installed if the initial nailing does not adequately reduce the rate of creep.

Acknowledgment

The writer wishes to express his appreciation to New Orleans District geotechnical design engineers Mr. Peter Cali and Mr. Mark Woodward for their assistance in providing information for use in this paper.

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13. ABSTRACT *(Maximum 200 words)*

Presented are the proceedings of the REMR Workshop on Levee Rehabilitation. The workshop was conducted to stimulate exchange of ideas and information regarding innovative methods for levee rehabilitation, directions for analytical and laboratory research, and possible field demonstrations of innovative methods. Presentations are made on seismic damage to levees, lime stabilization of levee slides, use of geogrids for levee slope repair, use of rock-fill trenches to stabilize levees, use of geotextiles for levee construction on soft soils, and soil nailing for slope repair. A copy of available written lectures is included in these proceedings.

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